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Paper 1396 October, 1957 PO 5

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KEMANO PRESSURE CONDUIT ENGINEERING INVESTIGATIONS[a]

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(Proc. Paper 1396)

ABSTRACT

This paper discusses some aspects of the basic investigations and tests associated with the design of pressure conduits for the world's largest underground hydro-electric generating station, with an ultimate 2,400,000 turbine hp installed capacity in 16 units. Two pressure conduits now in service are not exceeded in size, for equal head by any presently in existence.

INTRODUCTION

Kemano hydro-electric development, supplying Kitimat aluminum-reduction plant, is one of a series of notable Canadian hydro-electric works built by the Aluminum Company of Canada, Limited, the principal production subsidiary of Aluminum Limited. The general features have been described in several papers.(1) to (12)

Kemano is notable as the world's largest underground hydro-electric generating station, with an ultimate installed capacity of 2,400,000 turbine hp in 16 units. The pressure conduits, of which two of a probable four have been constructed and placed in service, are not exceeded in size, for equal head, by any built or under construction.

This paper deals with some aspects of the engineering investigations associated with the design and construction of the pressure conduits, of an unusual nature, which it is believed extend engineering knowledge of the interrelationship between a relatively thin steel liner inside the surrounding sheath of concrete backfill and rock mass through which the conduit passes.

Note: Discussion open until March 1, 1958. Paper 1396 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 5, October, 1957.

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Underground Power Plants

The first underground power plant was constructed about the turn of the century at Snoqualmie Falls, in the State of Washington. Since then over 100 such plants have been, or are being, built in locations all over the globe. Where the underground hydro-electric power plant meets the geologic and economic conditions, neither geography nor climate affect its use. Neither head, flow nor size limit development of waterpower resources by means of underground power plants. They have been built in all types of geological formations including igneous, metamorphic and sedimentary rocks. Many are built in sound, hard rock such as granite, requiring practically no support, while others are in badly fissured and even somewhat disintegrated formations, so that they require substantial and nearly continuous roof support.

General Features of Kemano Pressure Conduits

In the case of the 2,400,000 hp Kemano power plant, the powerhouse is emplaced in the mountain mass about 1400 ft. from the access portal. Initially one, 25 ft. equivalent diameter, 10.1 mile power tunnel (ultimately two) conveys the power water from the storage reservoir to the pressure conduits.

In the case of the first half of the ultimate 16-unit installation at Kemano, the pressure-conduit system (see Fig. 1) consists of the following elements, from the outlet end of the No. 1 power tunnel, proceeding downstream:—

1. A surge shaft excavated in the rock;
2. A steel wye, branching into two 11 ft. diameter conduits, each controlled by an 11 ft. diameter oil-pressure-actuated butterfly valve located about 300 ft. below the wye;
3. Just downstream from the butterfly valve on each pressure conduit, an air-inlet valve, to admit air into the pressure conduit whenever it is drained for inspection and maintenance;
4. Two 11 ft. diameter pressure conduit about 4,450 ft. long. The pressure conduits consist of a horizontal section at the upper end, a sloping run of 1200 ft. at an angle of 48° to the horizontal, an intermediate horizontal run and a further 2,000 ft. sloping run at 48° to the horizontal; at the lower end of the pressure conduit the manifold, symmetrically branched at 45° offsets and tapered to four 5 ft. diameter branch pipes, each serving one 150,000 hp generating unit.
5. About 140 ft. downstream from the first or major wye in the manifold, a 51 in. diameter spherical valve in each branch, located in an underground valve chamber, about 140 ft. from the center line of the generating units.

The sloping sections of the pressure conduits are located at an angle of 48° to the horizontal. The lengths of the horizontal runs were determined by this angle and the relationship of the natural profile of the ground and the requirements of rock cover above the conduit, fixed for design at 40% of the hydrostatic head at any point. Static head with full storage reservoir varies from about 200 ft. at the upper end to 2,590 ft. at the turbine nozzle level. Hydraulic studies showed that the design of the pressure conduits should provide for a maximum dynamic pressure of 2,850 ft. head or 1235 psi.

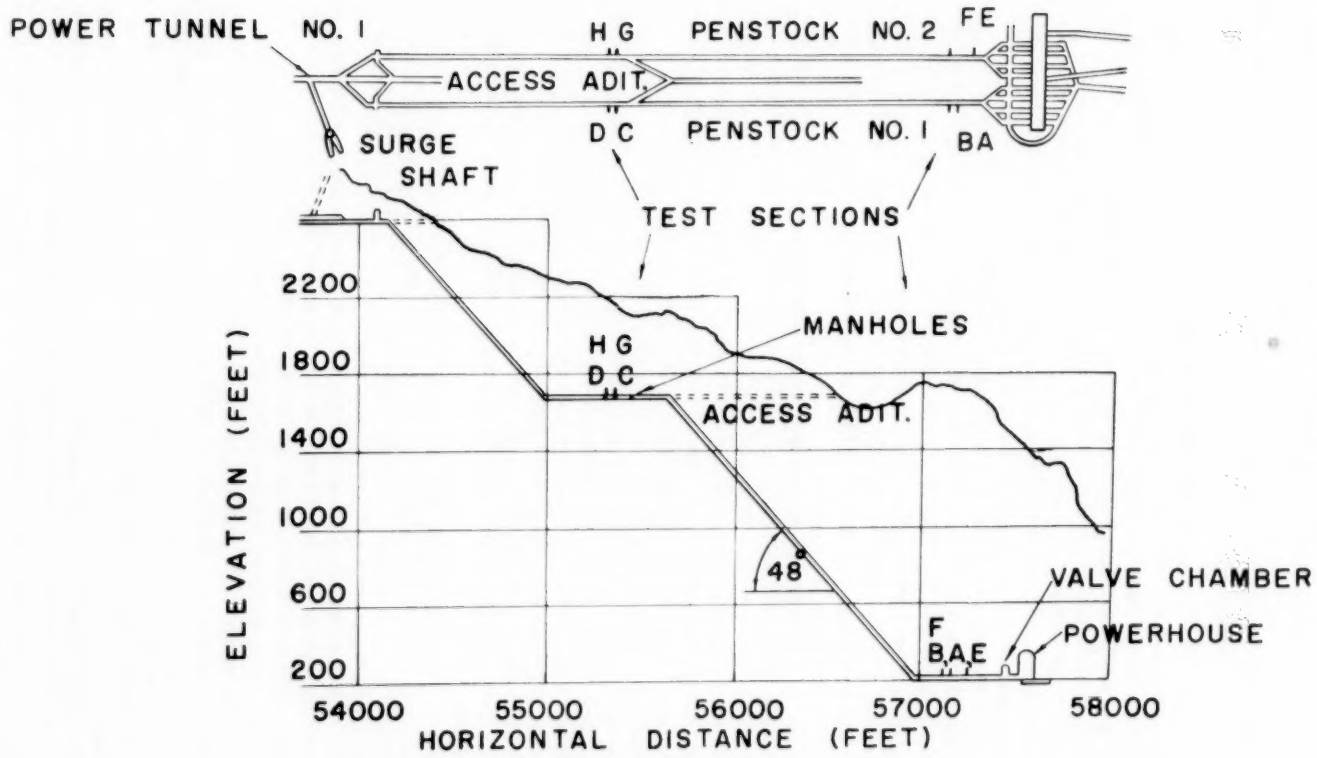


FIGURE 1

KEMANO PENSTOCKS NO. 1 & NO. 2

Reasons for Adoption of Pressure Conduits at Kemano

At Kemano the pressure conduits and the powerhouse proper are emplaced a considerable distance inside the mountain mass, for many reasons: —

1. Exposed steel penstocks for high-head power developments involved a maximum amount of steel as the penstocks must be designed to safely withstand indefinitely the normal maximum pressure plus pressure surges arising from sudden rejection or pick-up of load.
2. With suitable rock, a considerable reduction in the tonnage of steel could be secured, as the pressure conduit or conduits replacing the exposed penstocks only required such a thickness of steel liner that the steel and the encasing grouted concrete backfill and rock together could safely resist indefinitely the maximum stresses imposed in service.
3. Costly exposed penstocks could be replaced by a shorter pressure conduit or conduits, the discharge from the waterwheels being carried through free-flow (non-pressure) tailrace tunnels. Offsetting this were the costs of the tailrace tunnels, access tunnels to the underground power plant, and power cables between the underground transformers and the outdoor switchyard.
4. Massive supports and anchorages would be required for exposed penstocks; rock conditions are not always uniformly suitable for foundations.
5. Exposed penstocks, especially those in high-head developments, are vulnerable to enemy air action. Pressure conduits deep inside the mountain mass are not. An underground powerhouse, deep inside a mountain mass, would also be free from enemy air action.
6. Mountain slopes, heavily forested, presented a fire hazard arising from normal inspection and maintenance activities.
7. Should a fire occur on a penstock slope, the burnt-over slope would become vulnerable to potentially dangerous snow and rock-slides.
8. No maintenance of underground pressure-conduit external surfaces would be necessary but maintenance would definitely be required with exposed penstocks.
9. More economic foundations for massive powerhouse equipment could be secured, since the location at the foot of the steep slope rising from the valley floor had deeply filled with fluvio-glacial deposits and/or heavy talus which would have necessitated costly foundation work.
10. The marked advances since World War II in underground excavation and construction technique, particularly in the use of tungsten-carbide-tipped drill steel and bits, promised lower excavation costs.
11. Construction work on exposed penstocks on a steep mountain slope would be practically impossible during the winter season with a heavy treacherous snow cover, but it could be carried on throughout the year with underground pressure conduits. To a substantial extent work could be carried out on the power plant proper through severe winter conditions more expeditiously and more economically than with a surface plant.
12. The use of concrete in an underground power plant would be minimized. In any case, why substitute concrete for rock of better quality?
13. No structural maintenance would be required with the underground power plant.

14. More equitable working conditions would be available with the underground power plant, a factor of some significance as far as personnel are concerned.

Pressure-Conduit Design Considerations

While design of surface penstocks is founded on a rather definite basis, design of pressure conduits emplaced in a rock mass is much less tangible. The major problem is: What is the distribution of load between steel liner and surrounding rock? Design is influenced by many variables introduced by natural site conditions and construction techniques. Some of these are jointing, stratification, attitude and residual stresses of the rock formation; variations in strength and modulus of elasticity of the rock; shrinkage and imperfections of concrete backfill between liner and rock envelope; and variations in properties of steel liner, even though furnished to standard specifications. Experience has demonstrated that with a pressure conduit emplaced in a rock mass the rock acquires a permanent set under initial loading, with a tendency to modification of the contact between liner and concrete backfill under normal service conditions.

Design of pressure conduits is more of an art than a science at present, simply because we do not possess sufficient fundamental knowledge about the several elements involved. This is especially true of rock mechanics, so that Lord Kelvin's famous observation—

> "When you can measure what you are speaking about, and express it in numbers, you know something about it; but when you cannot measure it, when you cannot express it in numbers, your knowledge is of a meagre and unsatisfactory kind; it may be the beginning of knowledge, but you have scarcely, in your thoughts, advanced to the stage of science whatever the matter may be"

is particularly appropriate.

Professor E. L. J. Potts, of the School of Mines at Durham, England, has said:(13)

> "Too much has been written on the theory of strata pressures and movement, and too little has been done to measure actual values from which empirical reasoning can be carried out."

Many investigators have dealt with various aspects of the problems involved in the design of pressure conduits, notably Jaeger, whose comprehensive paper(11) provides an extensive bibliography. Others are Terzaghi and Richart who dealt with stresses in rock containing cavities;(14) Bleifuss who developed a theory(15) for the design of underground pressure conduits; and Vaughan who dealt with the design of steel linings for pressure conduits in underground hydro-electric plants.(16)

Brief consideration of the interaction between the steel liner and the surrounding sheath of concrete backfill and rock in a pressure conduit will be helpful. Assuming the liner is in intimate contact with the concrete sheath around it, which sheath replaces the excess rock necessarily removed during construction, the interaction is about as follows:—

1. The internal pressure will cause the steel liner to increase slightly in diameter, under the hoop stress set up, and tend to shorten

longitudinally; being restrained, longitudinal and transverse stresses will be set up in the girth welds at joints.

2. Expansion of the steel liner will transfer some load to the concrete sheath, thence to the rock; at the same time hoop stresses will develop in the concrete sheath. If these hoop stresses exceed the low tensile strength of the unreinforced concrete it will develop minute cracks.
3. That part of the internal hydrostatic pressure load transferred to the rock immediately adjacent to the sheath will cause it to plastically deform to some unknown degree. This will continue until "packing" under load has taken up the fissure and void spaces which exist naturally or arise from shaft-driving operations and are not completely filled with grout. Some minute cracking of the rock sheath can be anticipated in the vicinity of the pressure conduit since the tensile strength of rock is moderately low, compared with its compressive strength.
4. When the steel liner, surrounding grouted concrete backfill and rock sheath reach equilibrium under normal operating and maximum surge pressures, the stress in the steel should not exceed a "safe" value under the worst conditions of corrosion and/or corrosion fatigue foreseeable.
5. When the pressure conduit is drained for maintenance work, the steel liner, if it has not been stressed previously beyond the elastic limit and undergone sufficient permanent deformation, will decrease in diameter more than the surrounding grouted concrete backfill and rock sheath which has previously undergone a permanent plastic (packing) deformation plus some elastic deformation. Obviously the smooth external periphery of the liner will tend to separate, to some extent, from the surrounding sheath. Such separation gap, through the fissures and hairline cracks in the grouted concrete backfill and rock sheath, will fill with ground water under such pressure heads as exist in the mountain mass through which the pressure conduit runs.
6. Provided such external pressure heads are insufficient to cause inward buckling, refilling of the conduit and its return to service result in the steel liner and surrounding sheath resuming its loaded state as before.

The proportion of load from internal hydrostatic pressure transmitted by the liner to the rock can be calculated if the moduli of elasticity of the steel and of the rock are known, the elastic deformation of the liner in the radial direction being equated to the radial deformation of the rock. The principal difficulty with this procedure is lack of knowledge as to the real value of the modulus of elasticity of the rock formation traversed by the pressure conduit. The important consideration is that the stresses in the liner, under any service condition whatsoever, remain below the ultimate strength of the steel.

Design Aspects of Kemano Pressure Conduits

The early project estimates were predicated on one 15 ft. diameter, thin-walled pressure conduit for each power tunnel and each eight units. It was however felt that this would be too great a departure from normal practice and experience with underground pressure conduits. Accordingly, two 11 ft. diameter pressure conduits, with thicker liners were adopted. Even this design was substantially in advance of anything hitherto attempted.

On the basis of extensive engineering studies, it was decided that ductility

and weldability were essential properties for the steel liner to be used with the first pressure conduit to be built. Design and installation plans for the liner recognized that uniformity of support for the liner, and elimination of stress raisers and freedom from brittle fracture were of paramount importance. Voids or other lack of uniformity in the backfill concrete, physical break in the surface of the liner, and local application of heat to the steel without stress relieving were considered as potential weaknesses, to be avoided by all possible means. The steel finally selected was:—

Less than 1 in. thickness -- ASTM A285, Grade B, Firebox quality

One inch and greater thickness -- ASTM A201, Grade A, Firebox quality

Liner thicknesses adopted were based on a compromise among many factors, including retention of shape during handling, resistance to hydrostatic pressure with the conduit empty and limitations imposed by field welding. Emptying of the conduit would occur infrequently and could be done slowly. A permanent drainage system in the space between the liner and the rock envelope was provided. Since the reliability of this drainage system was considered questionable by some, it was decided that the liner thickness selected should withstand an external hydrostatic pressure equivalent to the vertical distance from the centre line of the conduit to the ground surface.

The minimum thickness of liner was taken at 9/16 in., for convenience in handling without distortion; the maximum thickness was 1-15/16 in. to facilitate manual welding from one side only. Between these limits, the thicknesses of liner plate were increased in increments of 1/16 in. down to a point near the manifold, where provision was made for welding from both sides, with thicknesses increased to 2-1/4, 2-1/2 and 3-1/8 in. at the field junction with the upstream wye. From this point downstream the proportion of the load assumed to be carried by the rock was decreased until the liner carried the entire hydrostatic load at normal free pipe stresses, which resulted in 2-3/4 in. thickness for the 5 ft. diameter turbine branches.

It can be seen that the basic design of the pressure conduit involved the use of a steel liner, serving both as a watertight diaphragm and as an inside form for the concrete backfill placed between the steel liner and the rock sheath. Under operating conditions, the hydrostatic pressure in the conduit is carried partly by the steel and partly by the weight of the overlying rock. At the powerhouse and valve chamber openings, the steel carries the entire stress arising from the internal water pressure. The concrete backfill and rock encasing the steel liner is thoroughly grouted at suitable pressures to fill all voids and seams, to so consolidate the rock that a thoroughly reliable and uniform structure would be secured.

Properties of Kemano Rock (Based on Samples)

It has long been recognized that there are extensive differences between the physical properties of a rock mass as determined from small samples and from large-scale tests.

At Kemano, laboratory measurements of compressive strength and modulus of elasticity were made on selected 2-1/8 in. diameter core specimens taken from diamond-drill holes associated with certain of the instrumentation utilized for the sphere test, which is discussed later.

Petrographic examination of four cores showed the rock in the vicinity of the sphere test, taken as representative of the structure through which the conduits would run, was fresh, sound, medium-grained, gray biotite-hornblende-quartz diorite composed of 40 to 60% hornblende and biotite, 25 to 50% plagioclase feldspar and 10 to 15% quartz with a trace of chlorite and zircon. An imperfectly developed gneissoid structure was noted, probably not sufficient to influence the compressive strength on axes normal or parallel to the orientation.

Eight samples were selected for sonic tests, utilizing the procedure specified in "Tentative Method of Test for Fundamental Transverse Frequency of Concrete Specimens for Calculating Modulus of Elasticity (Sonic Method), ASTM designation C215-47T," the fundamental frequency being determined by means of an oscilloscope. One of the eight samples was rejected because of a full length crack. The sonic modulus of elasticity was found to range from 4.73 to 8.68, averaging 6.20×10^6 psi. The sonic test provides a valuable index of rock quality but since it corresponds to the elastic properties of the rock under the no-load condition, it is only of qualitative value.

Four 2-1/8 in. diameter cores were selected for compressive tests to determine the modulus of elasticity under repeated loads. The ends of approximately 6 in. lengths of these cores were carefully polished to true parallel surfaces and the cores were then placed in a compression testing machine with electric resistance gauges attached to measure both vertical and circumferential deformations, with dial gauges to check vertical deformation. The modulus of elasticity and Poisson's ratio for the cores were found to be:

Core sample No.	Modulus of elasticity psi	Poisson's ratio at min. load	Poisson's ratio at max. load
1	7,820,000	0.140	0.30
2	5,470,000	0.125	0.28
3	7,300,000	0.105	0.26
4	7,400,000	0.090	0.26
Average	6,997,500	0.115	0.275

Repetitive loadings indicated that the rock samples were, for all practical purposes, elastic.

Investigations of plastic flow consisted of applying a constant compressive load to rock cores utilizing carefully designed and fabricated equipment. The specimens were loaded by a heavy spring restrained by tie rods. The spring and the core were stressed to a predetermined value in the compression testing machine. The tie rods were then tightened and the device removed from the testing machine. The compressive loading applied to the sample was then maintained for 500 hours, at a unit stress of 2,200 psi, the maximum expected in service. No appreciable plastic deformation could be discerned from the dial gauges or from the electric resistance gauges cemented to the core. This finding resulted in the belief that plastic flow of the rock would not be an important factor in the pressure conduits under the expected service loads. Obviously fissures in the rock would be reduced in size when load was applied to the rock, some of this closure being permanent and equivalent to plastic flow in the rock mass.

Following the tests described above, four cores were tested to failure in

compression with the following results:—

Core sample No.	Ultimate compressive strength psi
1	25,900
2	16,700
3	23,300
4	24,600

The purpose in making the compression tests was to ascertain the quality of the rock and to serve as a correlation with rock from other locations.

Sphere Test

Early work on the design of steel-lined pressure conduits was based on empirical methods but more recently, particularly in European practice, design has been based on the method of theoretical elasticity. In elastic analysis the lined pressure conduit is treated as an elastic pipe in an elastic mass of great extent. With the boundaries of the rock mass remote, rock deformations can be calculated using the Lame formulas, assuming the rock mass to be a cylinder of infinite external radius. Relative loads taken by the liner and the rock are determined by the pressures required for equal expansion. However, this approach has been most effectively used when the properties of the rock mass are predetermined by pressure tests on cylinders of the same diameter as the pressure conduit, the cylinders being inserted into excavations in the rock and concreted into place. The properties of the rock so determined reflect the various effects of joints, zones of weaknesses and plastic deformation.

Extensive consideration was given to the theoretical basis for design of the Kemano pressure conduits. One analytical method investigated involves application of Poisson's ratio to calculated compressive stresses in the rock along the extension of the horizontal diameter of the pressure conduit; this indicates that conduit pressures are effective only a very short distance out from the liner. Under this assumption, the concrete sheath between liner and rock serves only as a strut through which a portion of the load is transferred outwards until it is balanced by internal stresses in the rock and the strain is reduced to zero. Another approach, found to be realistic for Kemano, is based on treatment of the rock mass surrounding the liner and concrete backfill as a thick-walled cylinder enclosing a "cracked" zone immediately adjacent to the liner, this cracked zone to be assumed to act as a series of columns.

Since it is widely used in European investigation and design practice, the full-scale cylindrical test chamber was considered for Kemano but it appeared that the ratio of cylinder length to diameter would be limited to a low value by economic considerations. Careful studies revealed a number of uncertainties in the interpretation of test findings, from such an installation, these being:—

1. The distribution of stresses within a crylindrical test section of reasonably economic length would be greatly influenced by the end effects, the actual distribution of which would be unknown.

2. The conditions of longitudinal constraint of the pressure-conduit liner would be difficult to reproduce in the test installation.
3. The radial dilation of the cylinder probably would not be uniform, so that measurements of water pumped into the cylinder could not be used directly to evaluate the maximum radial dilation.

After much study, a test sphere was therefore utilized, this having the advantage of eliminating the problem of end effects and small size. It permitted compact instrumentation, minimized the volume of water to be pumped into the sphere under pressure, and permitted direct determination of the average dilation from the volume of water pumped into the sphere.

One disadvantage of the test sphere is that, within the cracked zone about it, the radial compression varies inversely as the square of the radius while in the cracked zone around a cylindrical section the radial compression would vary inversely as the radius.

Consequently, as soon as an exploratory tunnel had been driven to the approximate location of the powerhouse early in the construction period, a hollow sphere, of 1/2 in. plate, 10 ft. in diameter, was concreted into a side adit and tested under pressures up to 3600 psi. Extensive instrumentation was employed to obtain a measure of the modulus of elasticity of the rock mass and of the action of a thin-walled pressure vessel when encased in concrete and rock. Despite some unsatisfactory results with the gauging methods used, the modulus of elasticity of the rock was determined to be about 2,000,000 psi. The steel wall of the test sphere did not rupture under the high hydrostatic load of 2.8 times maximum dynamic pressure occurring in the pressure conduit, a result which warranted the basic concept that a thin steel pressure-conduit liner, properly encased in concrete backfill and backed up by sound rock of sufficient thickness would form a safe watertight design at the high service pressures under which the Kemano pressure conduits would operate.

Brewer has reported the dilation measurements of the test sphere and the rock deformations in a paper[17] describing the instrumentation utilized and the various findings.

Brewer's conclusions, somewhat condensed, were as follows:—

"The resistance of the rock to applied loads may be separated into two components. One component appears to be entirely elastic while the other appears to be related to the speed of loading. In the early tests where the rate of load application was high, the rock resistance to deformation was higher than was the case later when the rate of load application was low. On all tests, the release of load was made at a much greater rate than the application of load. As a consequence, the velocity effect makes itself evident by bowing of the unloading curve. - - Continued application of a constant load results in rock creep. On the basis of very limited data, it appears that creep is continuous but that its rate decreases with time so that it does not pose a serious structural problem. In general the velocity effect of the rock raises the value of E_r (modulus of elasticity of rock, thick sphere theory) in cases of transient loads but lowers E_r in the case of long-time application. The rock exhibits an ability to recover its creep loss on the removal of load. This recovery, however, is slow and appears to be about the same rate as the initial creep under pressure. Considering the effects of creep and rock

anisotropy, the value of rock modulus, assuming thick sphere theory, appears to reach a minimum value of 1.94×10^6 psi at 30 years of continuous load application of 3,600 psi. Creep appears to vary little with increasing pressure. Through the continuous determination of water consumption by the sphere at increasing pressures, it was possible to compute the average sphere dilation by this means and to compare it with the average sphere dilation obtained from the electric strain gauges on the diametral rods - - . The disparity between the two methods, about 1500 psi, is attributed to leakage at the pump."

Tests on Kemano Pressure Conduit No. 1

Much useful information for designing the steel for Kemano pressure conduit No. 1 was obtained from the laboratory sample tests of the rock and from the sphere test which was designed to determine the effective properties of the rock in situs. In spite of these data, the actual stresses which would be developed in the steel liner were not known exactly, and since the ultimate development at Kemano would require several similar conduits, it was felt desirable to determine the actual stress in the steel by tests on the first one.

Consideration was given to types of gauges which might be used inside the steel liner and to others which might be used on the outside surface of the steel liner or which could be used to measure strains in the concrete adjacent to the steel. Among the types of external installations considered were electric resistance gauges or vibrating-wire gauges on the outside surface of the steel; or Carlson-Pirtz stress meters, Carlson strain meters or vibrating-wire gauges located in the concrete near the liner to measure the external radial pressure on the steel. These installations had the disadvantage that the gauges and electrical leads would have to be placed during installation of the steel liner and would be susceptible to damage, especially while aggregate was being placed. In addition, the proposed use of gauges in the concrete raised the problem of determining its moisture and temperature expansion, and of determining the effective modulus at the time of the test. The possibilities of using vibrating-wire gauges or electric resistance gauges on the outside surface of the steel were limited because it was decided at this time that welds of any kind or holes or other stress raisers which might be required in designing suitable methods of fastening the gauges or gauge protectors to the steel had to be avoided. In the case of external electric resistance gauges there was also the difficulty of avoiding, or accounting for, the local bending stresses which would be present if a soft elastic waterproofing material was used; this would in effect leave a cavity in the concrete backing the liner at the gauge location. It was felt that the use of gauges in the concrete and rock would yield important data for checking theoretical analyses; however, the scope of the test program was limited by the time available and by cost considerations. The final selection of gauge installation was made on the assumption that the most important factor was the stress in the steel liner and that the installation should be made in a minimum amount of time and with a minimum amount of interference with construction work. It was therefore decided that all measurements should be made using gauges installed inside the conduit after construction work was completed.

Two different schemes were considered for use inside the penstock: strain

gauges could be bonded to the steel to measure the steel surface strain directly, or the change in diameter of the conduit could be determined using diametral rods and a mechanical type gauge or soft metal inserts, or using strain gauges or Carlson joint meters with the rods. The advantages of electric resistance gauges in this application are partly balanced by the difficulties involved in waterproofing them to withstand pressures up to 1200 psi, and in bringing electric leads out through the liner at this pressure. Because of the difficulties involved in any system considered, it was felt necessary to check some measurements using two completely different systems. It was therefore decided that a mechanical diametral gauge should be developed to record the maximum change in diameter of the conduit, and also details should be worked out for an installation utilizing electric resistance strain gauges applied directly on the inside surface of the steel liner.

A considerable amount of work was done in the laboratory to work out details and to proof-test the proposed systems. Particular attention was given to such problems associated with installing SR-4 electric resistance gauges in the conduit as methods of waterproofing, effects of hydrostatic pressure on the gauges, zero drift, effect of long leads (to about 400 ft.), methods of anchoring leads inside the liner, and methods of bringing insulated leads through the liner and the concrete. Most of this work was done by testing the various components in a steel pressure tank at pressures up to 1500 psi. Work was also done to develop a workable diametral gauge and to devise a suitable method of attaching it to the steel liner.

The general layout of the first half of the development at Kemano is shown in Fig. 1. When the tests were made on penstock No. 1, the shaft for penstock No. 2 had been driven but the steel liner for it had not been placed. Pressure conduit No. 1 consisted of a 400 ft. long, 11-ft. diam., welded steel liner concreted into a shaft which was excavated to a minimum diameter of 15 ft. The thicknesses of the steel sections varied from 9/16 in. at the 2600-ft. elevation to 1-15/16 in. and thicker at the 210-ft. elevation. After each section of steel was placed and welded, prepakt aggregate was placed between it and the bedrock.(18) This aggregate was later pressure grouted through special holes located at 90-degree spacing around the conduit at sections 10 ft. apart. Two 4-in. pipe drain-headers were provided in the concrete and until grouting was completed these were connected to the conduit by means of 2-in. pipe nipples. These nipples were designed so that they could be withdrawn through drain holes in the steel liner thus leaving a path for ground water around the steel liner to flow into the drain. The grout holes and drain holes were sealed with threaded and welded plugs before the conduit was filled.

The system developed for the installation of electric resistance gauges involved attaching four circumferential gauges and one temperature gauge at a cross-section of the conduit near one of the drain holes in the liner. Leads from the gauges were to be waterproofed and bonded to the inside of the liner until they could be brought through the liner into the drain pipe by means of a specially designed plug which screwed into the drain hole. Once outside the pressure conduit, the lead wires were to be connected to a special multiconductor cable which would run inside the drain pipe to a location suitable for an instrument station.

In order to obtain accurate readings from the resistance gauges, they had to be well bonded to the steel and waterproofed to maintain a very high resistance to ground; in addition, the effects on the gauges and lead wires of pressure and temperature changes had to be considered. These corrections

are usually effected by mounting a "compensating" gauge on a stress free surface close to the active gauge and then taking measurements with these two gauges in opposing arms of a bridge circuit. In the present case, the steel liner was restrained against displacement in the axial direction, so it was decided that the active circumferential gauges should be balanced against longitudinally directed compensating gauges. The difference between having these gauges on strain-free, rather than on stress-free, surfaces was then to be accounted for in calculating the stresses from the readings. Temperature changes had to be measured to do this; however, under any system, temperature should be measured since such changes affect the stress in the liner.

The electric resistance SR-4 strain gauges were waterproofed using a thin layer of wax covered by a synthetic rubber (Thiokol) compound and following a procedure similar to that described by Dean.(19) Fine, 22-gauge, vinyl-plastic, insulated lead wires used inside the pressure conduit were bonded to the steel surface by laying them under a strip of extra thin Koroseal electrical tape and then bonding and sealing this using the synthetic rubber compound as used over the gauges. The seals between the internal high-pressure area and the drain headers were made by casting the wires in this rubber compound in small radial grooves around the top of a special steel plug which screwed into an unsealed "drain hole" in the steel liner as shown in Fig. 2.

The locations at which gauges were installed were chosen from consideration of many factors including the length of lead wires which could be used, the thickness of the steel relative to the pressure, the nature of the concrete and rock behind the steel, the locations of drain holes, and the ease of access to the gauges. Four sections were chosen as indicated in Fig. 1; sections A and B were located in the horizontal portion of the 1-15/16 in. thick, 11-ft. diameter conduit at the 210-ft. elevation, while sections C and D were in the horizontal 15/16 in. thick portion at the 1680-ft. elevation.

The second method used to determine the strain in the steel liner of the conduit, and subsequently to determine the stress, involved measurements of the change in diameter of the conduit as it was loaded. The average circumferential strain in the steel is equal to the average change in diameter of the section, divided by the diameter. A mechanical gauge consisting of a diametral pipe connected to a linkage which magnifies displacement approximately 30 times and records it as a scratch on an aluminum plate was developed to record the maximum change in diameter. A sketch of this gauge is shown in Fig. 3. These gauges were designed to fasten into special plugs put into grout holes which were spaced at 90-degrees around sections of the steel liner, so that two measurements were used to represent the average for a section. Mechanical gauges were installed at only two sections; one in the 1-15/16-in. plate near section A at the 210-ft. elevation, the other in the 15/16-in. plate near section C at the 1680-ft. elevation. Each gauge was checked and calibrated in the laboratory before it was installed.

Test Procedure and Results, Conduit No. 1

The programme of operations actually followed for the testing of conduit No. 1 was briefly as follows:

1. The strain gauges at the 219-ft. elevation were "zeroed" and the mechanical gauges were set, then water was pumped into the lower

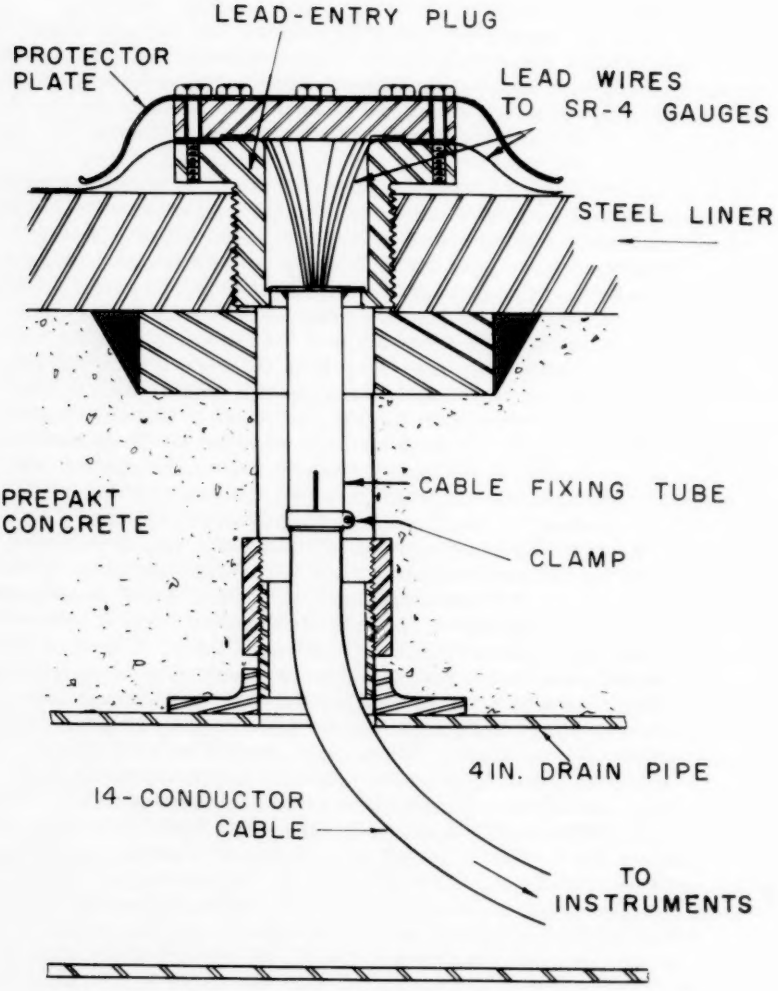


FIGURE 2

LEAD-ENTY PLUG IN DRAIN HOLE

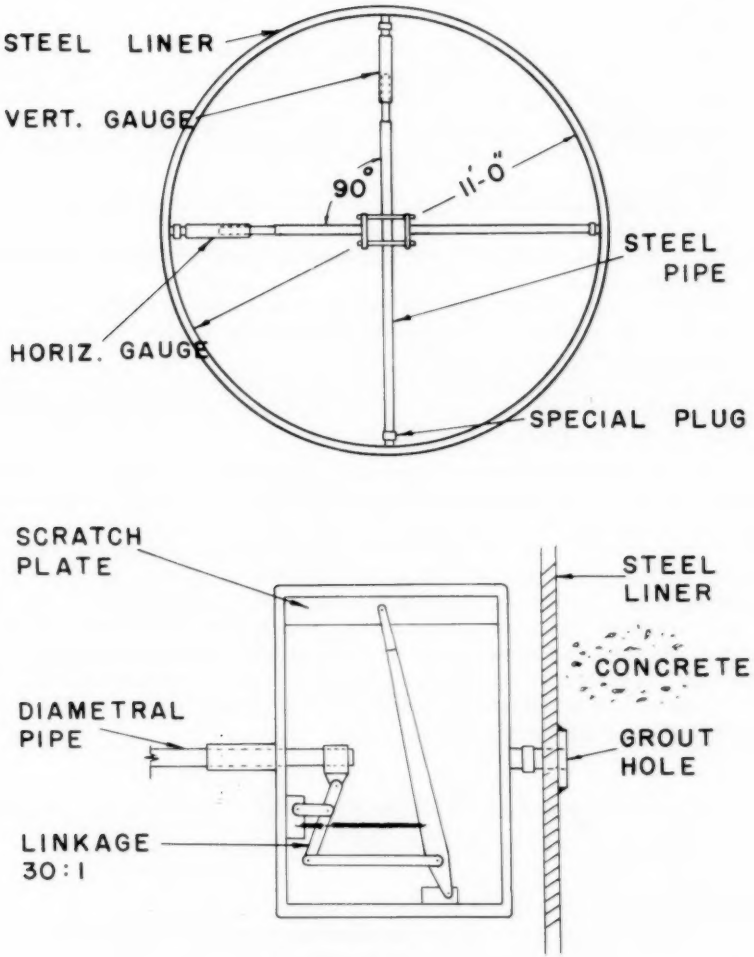


FIGURE 3

MECHANICAL GAUGE AND ASSEMBLY ARRANGEMENT IN PENSTOCK NO. 1

horizontal section of the conduit to provide a cushion for water which would later come down from the top end. This pumping continued for 24 hours at which time the horizontal section was two-thirds filled.

2. Strain gauges at the 1680-ft. elevation were "zeroed" and the mechanical gauges there were set.
3. Water was released through the intake at Tahtsa Lake in such a manner that the conduit filled to the 2590-ft. level in 1-1/2 hours. The power tunnel was not filled during the test.
4. Strain gauges and temperature gauges at both levels and pressure gauges at the 210-ft. level were read continuously while the conduit was filling.
5. The conduit was held under full test head for 10 hours. During this time the gauges were read periodically.
6. The conduit was drained in twenty minutes by opening the nozzles in two generator units.
7. Readings were taken at the 210-ft. elevation during draining, but because of the rapidity of the pressure drop at the 1680-ft. level, no significant readings were obtained there.
8. After emptying the conduit, the record plates were removed from the mechanical gauges. All gauges were then removed and temporary drain and grout-hole plugs were replaced.

In order to determine stresses from readings of the strain gauges, it was necessary to first interpret the observed data in terms of true circumferential strain. This true strain includes thermal strain, or expansion, as well as strain resulting from both circumferential and longitudinal stresses. Since the active gauges were balanced against unstrained, longitudinally-directed compensating gauges, the indicated strain was approximately equal to the true hoop strain. These indicated strains were corrected for two effects: first, because of the long lead wires, lead wire resistance was large enough to cause some error; second, the stress system is biaxial, consequently the cross-sensitivity of the gauges had to be taken into account. These corrections were made using the equation:

$$\varepsilon_\theta = \left(\frac{1-k\mu_m}{1-k}\right)\left(1+\frac{R_L}{R}\right)\varepsilon_o$$

in which,

ϵ_o = strain indicated by instruments set to the gauge factor specified by the gauge manufacturer.

μ_m = 0.285 = Poisson's ratio of material used by the manufacturer in calibrating the gauges.

K = cross-sensitivity/longitudinal-sensitivity of the gauges used.

R = resistance of the active gauge.

R_L = resistance of the lead wires to the active gauge.

The circumferential stresses, σ_θ, were determined from the true strains, ϵ_θ, and the temperature drop, T, as follows:

$$\sigma_\theta = \frac{E}{(1-\mu^2)}\varepsilon_\theta + \frac{EBT}{(1-\mu)}$$

in which

E = $29(10)^6$ psi = modulus of elasticity of steel.

μ = Poisson's ratio.

B = Coefficient of thermal expansion of steel.

T = Temperature change in the steel, taken positive for a drop in temperature.

The longitudinal stress in the steel then would be given by:

$$\sigma_z = \mu\sigma_\theta + EBT$$

The mechanical diametral gauges installed in the conduit responded to the change in length of the diametral rods caused by temperature changes as well as to changes in diameter of the conduit due to both temperature and pressure. Consequently, if the average of the horizontal and vertical gauge readings is used to represent the average change in diameter of the conduit, then the true average strain, including thermal strain, is:

$$\varepsilon_\theta = \frac{1}{2D}\left(\frac{\ell_H}{M_H} + \frac{\ell_V}{M_V}\right) - BT$$

in which

D = average diameter of the conduit liner.

ℓ_H, ℓ_V = lengths of scratches on the record plates from the horizontal and vertical gauges.

M_H, M_V = magnification factors of the horizontal and vertical gauges.

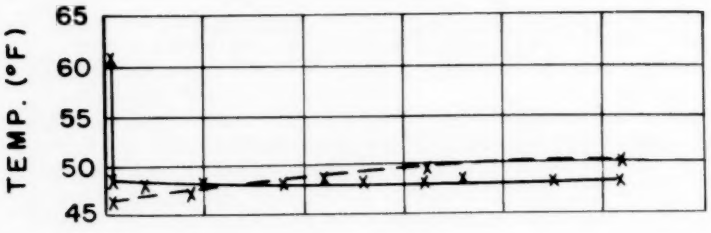
B = thermal coefficient of expansion of the rods and gauge mechanism.

T = change in temperature of the rods and gauges; taken as positive for a drop in temperature.

The stresses were calculated from this average strain in the same way as was done for the electric resistance gauge results.

The average temperatures and the average stresses determined from the strain gauge readings are shown in Fig. 4 and 5. Two gauges at the 1680-ft. elevation developed faults during the tests, consequently the results at this elevation were based on three operating gauges at each of sections C and D. All gauges were operating at sections A and B.

Strain gauge readings taken prior to filling the penstock, particularly those taken while water was being pumped into the lower end, indicate that at the gauges, there were sufficiently large tensile stresses to cause positive strains which more than offset the thermal contraction of the steel. From



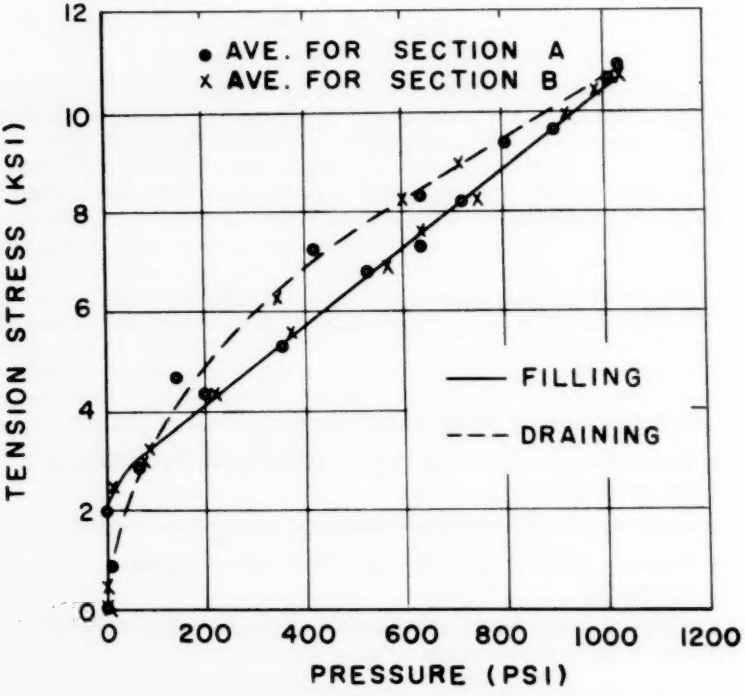
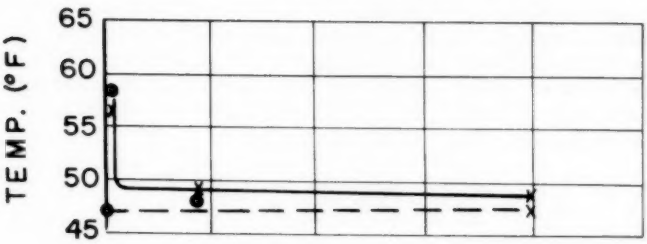


FIGURE 4

STRESS AND TEMPERATURE IN THE $1\frac{15}{16}$ IN. STEEL AT THE 210 FT ELEVATION OF PENSTOCK NO. 1



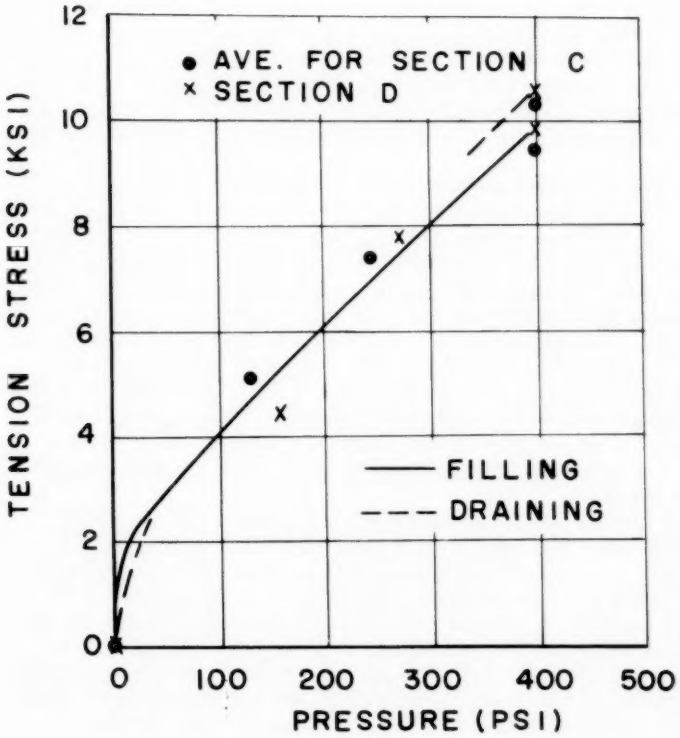


FIGURE 5

STRESS AND TEMPERATURE IN THE $^{15}/_{16}$ IN. STEEL AT THE 1680 FT ELEVATION OF PENSTOCK NO. 1

Fig. 4 it can be seen that this stress is about 2000 psi for a temperature drop of about 13°F. This stress was thought to be largely due to the temperature difference between the inside and outside surfaces of the steel liner. If the inside temperature of a thin-walled cylinder is T degrees lower than the outside temperature, then the circumferential stresses at the surfaces according to Timoshenko and Goodier[20] are approximately:

$$\sigma_\theta = \frac{BET}{2(1-\mu)}$$

Consequently, the tensile stress on the inside face of the steel liner would be about 135 psi per degree Fahrenheit temperature difference. Assuming the concrete temperature to be 13°F above the water temperature, the stress in the steel due to this temperature gradient would be 1.75 ksi, so that the stresses observed before the pressure built up could be due to this effect. However, later measurements on conduit No. 2 indicated that this thermal gradient is considerably smaller than 13°F. It is also possible that there is some local longitudinal thermal contraction which would affect the compensating gauges and therefore show up as a tension stress when it is attributed to the active gauge.

The average change in stresses observed during filling of the conduit appear to be very nearly linear; however, higher stresses were indicated during draining than were observed during loading. Apparently, due to cracking, creep, or plastic deformation, the rock and concrete offered less support to the steel during unloading than it did during loading. In fact, the rapid stress drop at low pressures indicates that the steel was virtually free from the concrete at this stage. The separation of the steel from the concrete can be partially explained by the contraction of the steel caused by temperature drop; however, residual deformation of the concrete or rock is indicated. The thermal stresses present before the conduit was pressure tested seem to have partly disappeared after the conduit was drained. This may be due to the stabilization of temperatures and reduction in the thermal gradient through the thickness of the steel.

In this test of Conduit No. 1, the maximum test pressure at the 210-ft. elevation was 1040 psi. The full dynamic head for which the conduit is designed is 1235 psi, consequently the test results must be extrapolated to determine the stresses which would exist under this head; these figures are 12,300 psi in the 1-15/16 in. steel at the 210-ft. elevation and 14,900 psi in the 15/16 in. steel at the 1680-ft. elevation. If these same thicknesses of steel had been used without the concrete and rock backing, the circumferential steel stresses would have been 42,600 psi and 42,200 psi respectively. It is evident then that the concrete and rock have reduced the steel stresses by 71% and 65% at the 210-ft. and 1680-ft. elevations respectively.

The results obtained from the mechanical gauges are shown on the table on the following page. The stresses determined from readings of these gauges at the 210-ft. elevation agree reasonably well with those determined from readings of the electric strain gauges. The changes in the two diameters at this test section were not the same, that of the horizontal diameter being only about 70% as large as the change in the vertical diameter. The results obtained from the mechanical gauges at the 1680-ft. elevation are somewhat

Gauge Location	Maximum Pressure (psi)	Scratch Length (in.)	Average Total Strain (microin./in.)	Circumferential Stress (psi)
210'El.	1040	1.73 (Vertical) 1.18 (Horizontal)	268	12,030
1680'El.	400	0.89 (Vertical) Horiz. gauge, faulty	129	7,600

lower than results obtained with the electric resistance gauges. This is probably largely because one gauge did not function properly so that the average change in diameter of the section had to be assumed equal to the change in the one vertical diameter.

Tests on Kemano Pressure Conduit No. 2

Soon after Kemano conduit No. 1 was put into service, the decision was made to proceed with the lining and concreting of conduit No. 2 using the same design and installation methods as followed on No. 1. Because it was felt that the concrete or rock conditions might be somewhat different, and because there was very little information about the effect of rock creep, consideration was given to test work for this second conduit.

It was decided that electric resistance gauges should again be installed inside the conduit as an independent check on results obtained in the first tests and for correlation with other measurements which would be made in this second test. Consideration was again given to gauges which could be used externally on the conduit. Such gauges have the advantage that they would be a permanent installation so that, provided they had good enough zero stability, measurements could be made periodically to check the effect of creep in the rock or concrete or to detect changes from other causes. The use of gauges in the concrete and rock could also provide important data for checking theoretical analyses; for example, an analysis based on a "cracked-zone" surrounded by an infinite elastic mass.

The requirements which were considered necessary in gauges for installation in the concrete or rock are: the gauges would have to be read from a station about 300 or 400 ft. from their installed position; the gauges and any supports or waterproofing should have approximately the same effective modulus as the concrete or rock so that the stress distribution in the vicinity of the gauge would be the same as if the gauge was not there; and the gauges should preferably have high strain sensitivity, low temperature sensitivity and good zero stability. The type of gauge which seemed to best meet all of these requirements was the vibrating-wire or "acoustic" strain gauge.(21,22)

In principle the vibrating-wire gauge consists of a stretched steel wire held at its ends by the material or member under test and made to vibrate at its natural frequency. If the material suffers a change in strain, the wire undergoes a proportionate change and a measure of this strain is given by the change in frequency of vibration of the wire. The wires are plucked electrically and their frequency is picked up by a small coil in the gauge. By tuning a calibrated reference wire against the measuring-gauge wire, changes in

strain in the member under test are determined directly by differences in a scale reading. Various forms of these gauges are available. The types made for embedment in concrete consist of a tube with flanged ends; typical dimensions of these gauges vary from about 1/4 in. diam. by 4 in. long to 1-1/2 in. diam. by 10 in. long. These gauges are designed so that their effective modulus is approximately the same as that of concrete. Other gauges are made to fasten to lugs on the surfaces of members under test; one gauge of this type consists of a tube with an extension bellows so that the wire is sealed in a tight container which, with the test member, can be immersed in water or sealed in concrete. Gauges operating on the vibrating-wire principle are also available for measuring temperature and hydrostatic pressure.

After some laboratory work with these gauges, proposals were made to install different types to measure the circumferential strain on the outside surface of the steel, the radial strain in the concrete and rock, and temperature and hydrostatic pressure at both the 210-ft. and 1680-ft. elevations. However, before this proposal was implemented, field work on the conduit had progressed so far at the 210-ft. elevation that it was impossible to follow the original test plans there. The installation as finally made included four circumferential gauges on the steel and four radial gauges in the concrete close to the liner at one section, section E, at the 210-ft. elevation, and at two sections, sections G and H, at the 1685-ft. elevation. In addition, radially directed gauges were located on two diameters at section G, at distances of 2 ft., 4 ft. and 8 ft. from the outside of the steel. A temperature gauge, two manometric gauges and two "compensating" vibrating-wire gauges were also used at each of these three sections. Electric resistance strain gauges were installed at a fourth location, section F, as well as at sections E, G and H. The general locations of these four sections are shown in Fig. 1. At section E the steel is 2-1/4 in. thick, at F it is 1-15/16 in. thick and at G and H it is 15/16-in. thick. A cross-sectional view of the 11-ft. diameter conduit showing the locations of all of the gauges is given in Fig. 6.

The gauges used to measure the circumferential strain on the outside surface of the steel were Télémac type SB-90 vibrating gauges. This type of gauge consists of a 0.010 in. diameter wire 90 mm. long inside a tube which has a bellows inside the head at one end. The heads of this covering tube fastened into steel lugs which were welded to the liner; because of the bellows, the initial tension in the vibrating-wire can be set to any desired initial tension. After being mounted and adjusted to a predetermined "zero," each gauge and its attaching lugs were wrapped with heavy paper and tape to prevent concrete from bonding to it or restricting free motion of the bellows. The gauges were then covered with moulds in the shape of half-cylinders and these were packed with concrete; this concrete provided protection to the gauges while the prepakt aggregate was being placed.

The vibrating-wire strain gauges used in the concrete and rock were Télémac type B-90 gauges. These are similar in size to the SB-90 gauges used on the steel but the protecting tube is rigid and is flanged at the end to provide good fixity in concrete. The initial "zero" is preset by the manufacturer. Before installing these gauges around the conduit, each gauge was cast in a concrete cylinder in the laboratory. This cylinder served to protect the gauge in the field and permitted a direct calibration by test loading the cylinder in a testing machine in the laboratory. Gauges which were to be

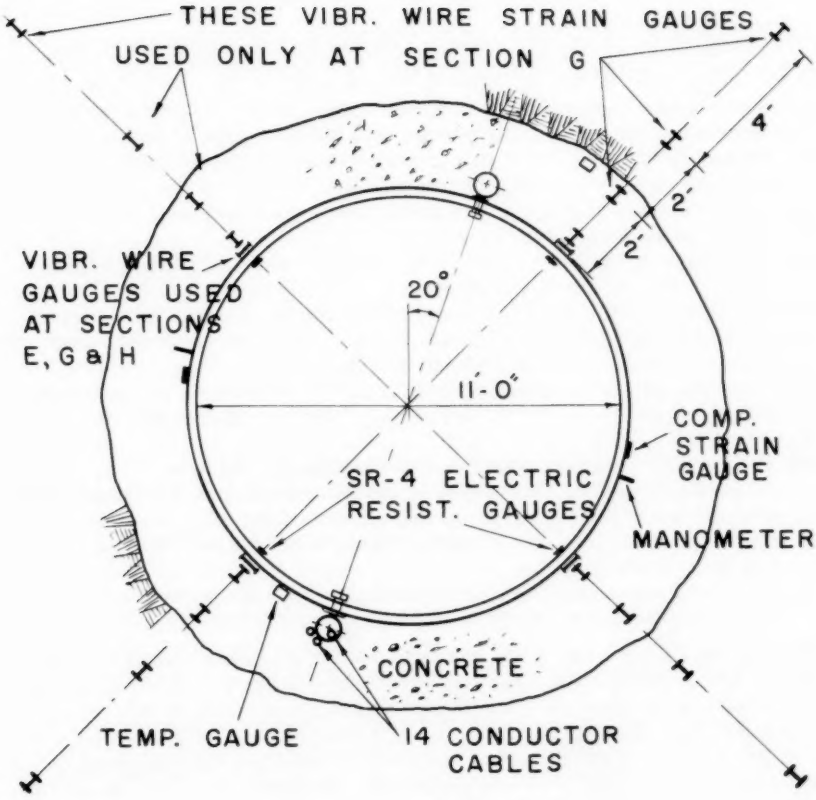


FIGURE 6

CROSS-SECTION OF KEMANO PENSTOCKS
SHOWING GAUGE LOCATIONS

embedded in the rock were cast in extra small concrete cylinders so they could be placed in 3-in. diameter diamond drill holes. The concrete mix used in all of these blocks was selected after considerable experimentation because the modulus had to be the same as that expected in the prepakt concrete and excess shrinkage which would affect the gauge "zero" had to be avoided.

Test Procedure and Results, Conduit No. 2

The program of operations actually followed for the testing of conduit No. 2 was very similar to that used in the tests on the first conduit:

1. The resistance strain gauges and the external vibrating-wire gauges were read, then water was pumped into the lower end of the conduit for about 24 hours, after which time the horizontal section was filled and under a pressure of about 40 psi. This pressure was held for about 45 hours and the gauges at this lower elevation were read periodically during this time.
2. All gauges at the 1680-ft. elevation were "zeroed" or read and the manhole at that elevation was sealed.
3. The power tunnel was full, so water was put into conduit No. 2 by partially opening a valve in a bypass line so that filling would take about 9 hours. Maximum test pressure was 1100 psi.
4. All strain gauges and temperature gauges at both levels and pressure gauges on the conduit at the 210-ft. level were read periodically while the conduit was filling.
5. The conduit was held under full test head for about 13 hours.
6. By partially opening the nozzles in one generating unit, the conduit was drained in about five hours.
7. Readings were taken on all gauges while the conduit was being drained and after it was empty.
8. The electric resistance gauges installed inside the conduit were removed and temporary drain-hole plugs were replaced.

In planning for the tests it had been assumed that two instruments would be available for reading the vibrating-wire gauges at the two different levels. Unfortunately, only one such instrument was available so that it was found necessary to read the gauges at the 210-ft. elevation from an instrument station at the 1680-ft. elevation. This was done by using a spare telephone circuit; however, there was considerable outside interference on the line so that the number of readings taken this way was limited, particularly during draining of the conduit.

Readings of the electric resistance strain gauges were interpreted in terms of stress in the steel liner using the same equations and procedure as were used in the test on the first conduit. A similar procedure was followed in interpreting the results from the vibrating-wire gauges attached to the outside of the steel. However, with these gauges the only correction made to the indicated strain, to get the true total strain, including thermal strain, was a temperature correction, the magnitude of which was determined by some tests in the laboratory.

To determine the radial compressive stress in the concrete surrounding the steel liner from readings of the B-90 vibrating-wire gauges, the modulus of elastivity of the prepakt concrete had to be known. This was

determined from tests made in the laboratory on 6-in. diam. by 12-in. long cylinders which were pressure grouted at the same time that the section in question was done. It was found that the modulus of elasticity of the concrete generally increased slightly with age. At the date of the test, the modulus determined for the concrete at the 210-ft. level was $3.6(10)^6$ psi while that for the concrete at the 1680-ft. level was $3.1(10)^6$ psi. The effective modulus of elasticity of the rock mass cannot be determined experimentally in the laboratory, consequently the strains indicated by the gauges in the rock at section G could not be directly interpreted in terms of stress.

The average steel temperatures and the average stresses determined from the resistance strain gauge and vibrating-wire strain gauge readings are shown in Fig. 7, 8 and 9. The average radial stresses and the temperatures in the concrete just outside the steel liner at each elevation are given in Fig. 10 and 11, and the average radial strain in the concrete and rock at different distances from the steel at section G is shown by Fig. 12. All of these graphs were obtained by averaging the readings of similarly located gauges so that most curves represent the average of about four gauges. Before and during the test a number of gauges of all types, (about 25%), became inoperative due to poor waterproofing or for various other reasons; readings of these gauges therefore could not be included.

In general the resistance strain gauge results from this test on conduit No. 2 confirm the results obtained at similar sections in the tests on conduit No. 1. The stresses in the 1-15/16-in. thick steel at section F, Fig. 8, are almost identical to those observed at sections A and B, Fig. 4, and the stresses in the 15/16-in. thick steel at sections G and H, Fig. 9, agree very well with those observed at sections C and D, Fig. 5. This agreement covers the initial thermal effects as well as the filling and draining. Extrapolating these results, as was done with the results for conduit No. 1, the stresses which would exist under the full maximum design pressure of 1235 psi are: 12,300 psi in the 1-15/16-in. steel at section F and 13,000 psi and the 15/16-in. steel at the 1680-ft. level. These electric resistance gauge results thus indicate that the concrete has reduced the stresses in the steel at these locations by 71% and 69% respectively.

For some reason at section E, Fig. 7, the stress in the steel is higher than it is at section F even though the hydrostatic pressures are the same and the steel at E is 2-1/4 in. thick as compared to 1-15/16 in. at F. The electric resistance gauges at section E indicate that the steel stress would be 16,500 psi under a hydrostatic pressure of 1235 psi or that the concrete has reduced the steel stresses by only 55%. The vibrating-wire gauges on the steel at this section show this stress to be 14,000 psi; based on this figure, the concrete carries 62% of the internal pressure. The gauges are therefore not in exact agreement; however, from either set the indicated stress in the 2-1/4-in. thick steel is significantly higher than that in the 1-15/16-in. steel at the same elevation.

The radial stresses in the concrete, given in Fig. 10 and 11, also show that the concrete at section E is taking a considerably smaller portion of the load than at other sections. Considering that these B-90 gauges are in the concrete about 4 inches away from the outer surface of the liner, and extrapolating the results to the full design pressure of 1235 psi, these gauges indicate that the concrete at section E carries only 50% of the applied hydrostatic pressure while at section G and H it carries 72%.

The radial strain in the concrete and rock behind the 15/16-in. thick steel

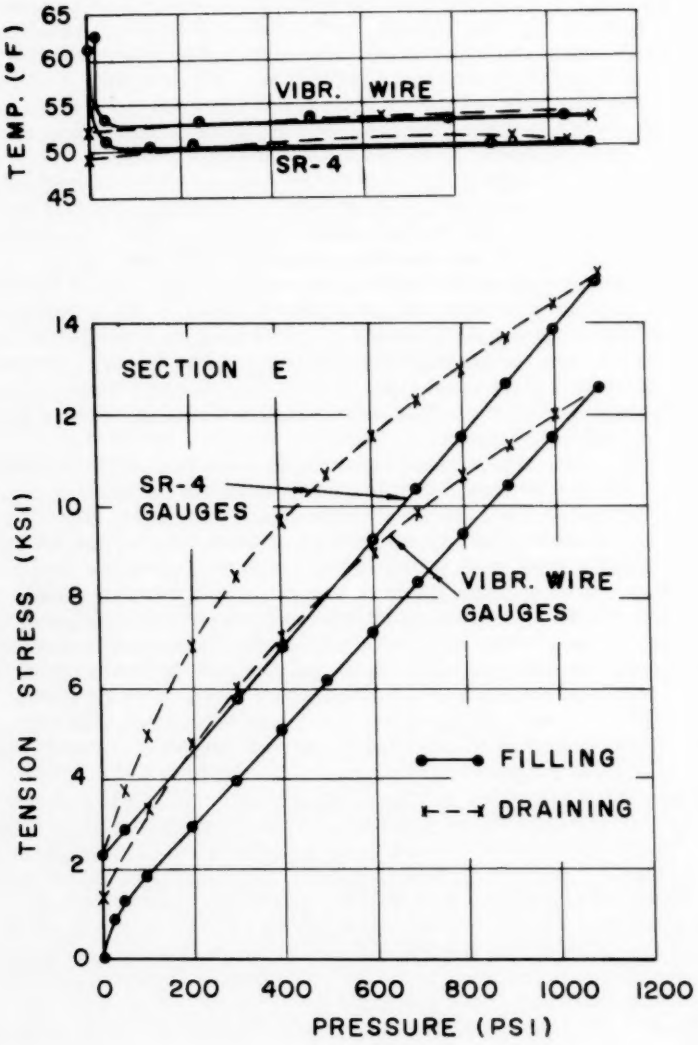


FIGURE 7
STRESS AND TEMPERATURE IN THE 2 ¼ IN. STEEL AT THE 210 FT ELEVATION OF PENSTOCK NO. 2

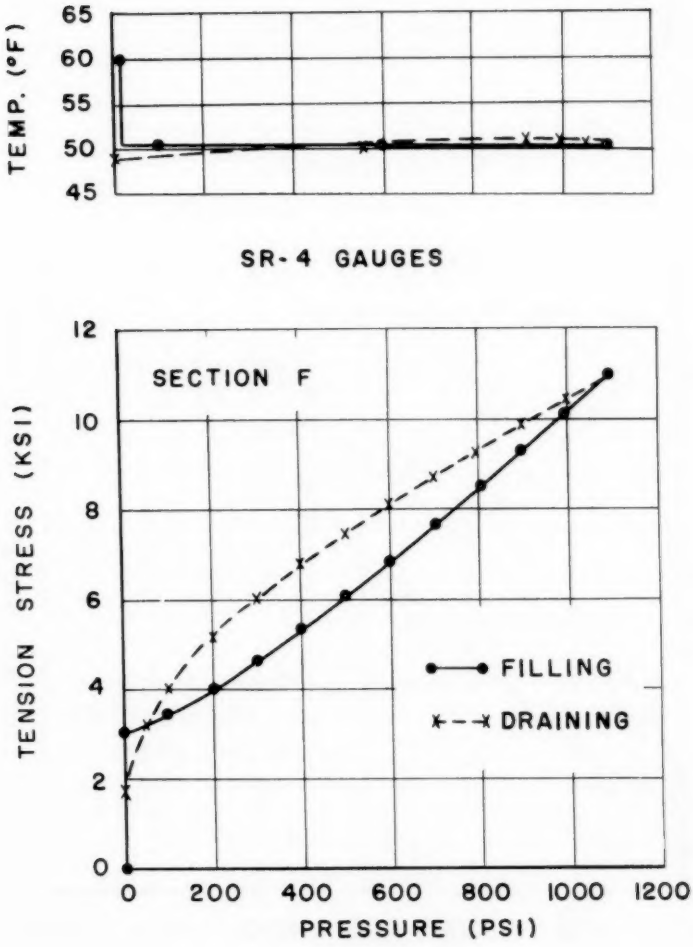


FIGURE 8

STRESS AND TEMPERATURE IN THE $1\frac{15}{16}$ IN. STEEL AT THE 210 FT ELEVATION OF PENSTOCK NO. 2

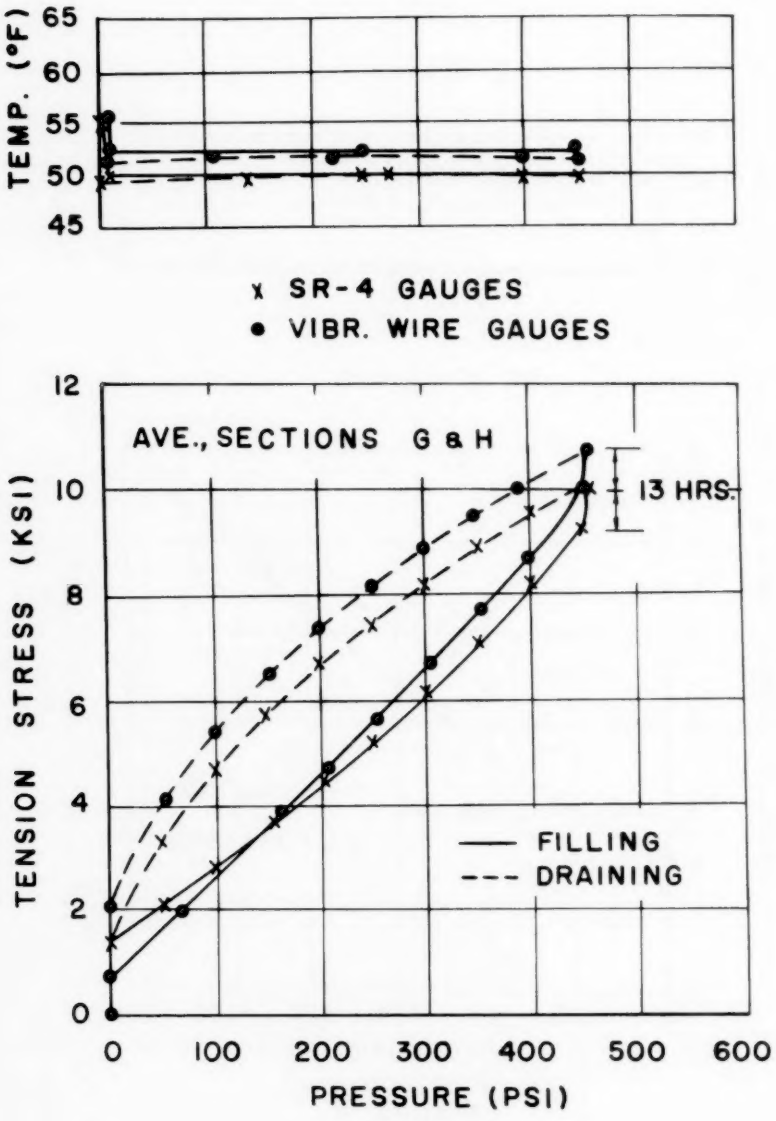


FIGURE 9

STRESS AND TEMPERATURE IN THE 15/16 IN. STEEL AT THE 1680 FT ELEVATION OF PENSTOCK NO. 2

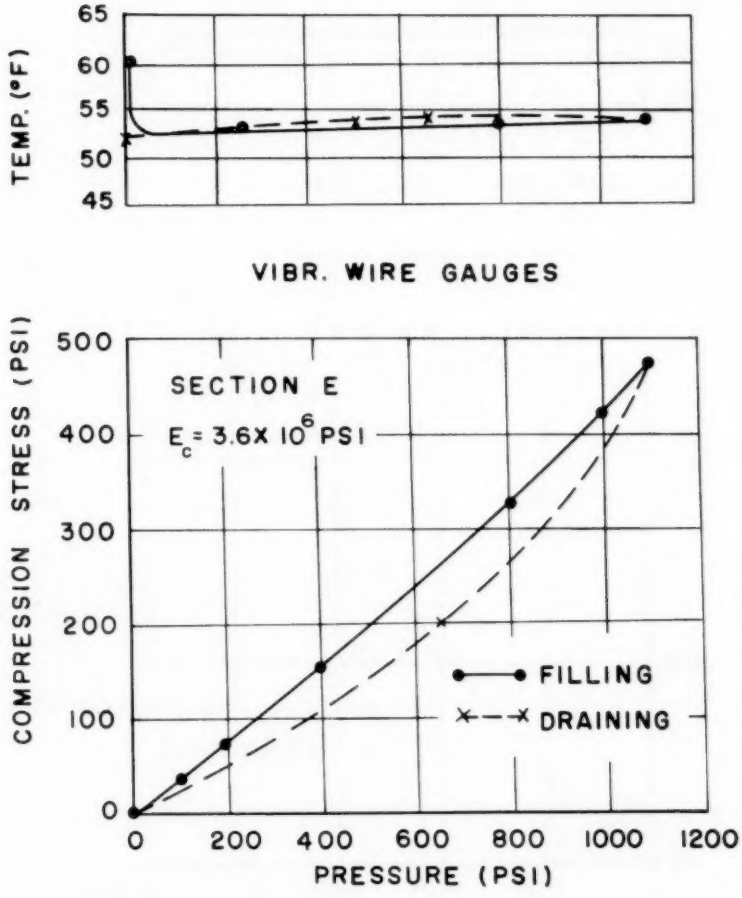
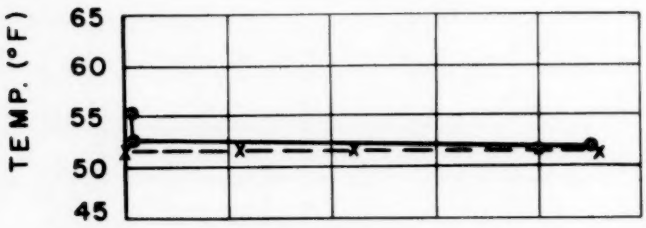


FIGURE 10

RADIAL STRESS IN THE CONCRETE OUTSIDE THE 2 ¼ IN. STEEL AT THE 210 FT ELEVATION OF PENSTOCK NO. 2



VIBR. WIRE GAUGES

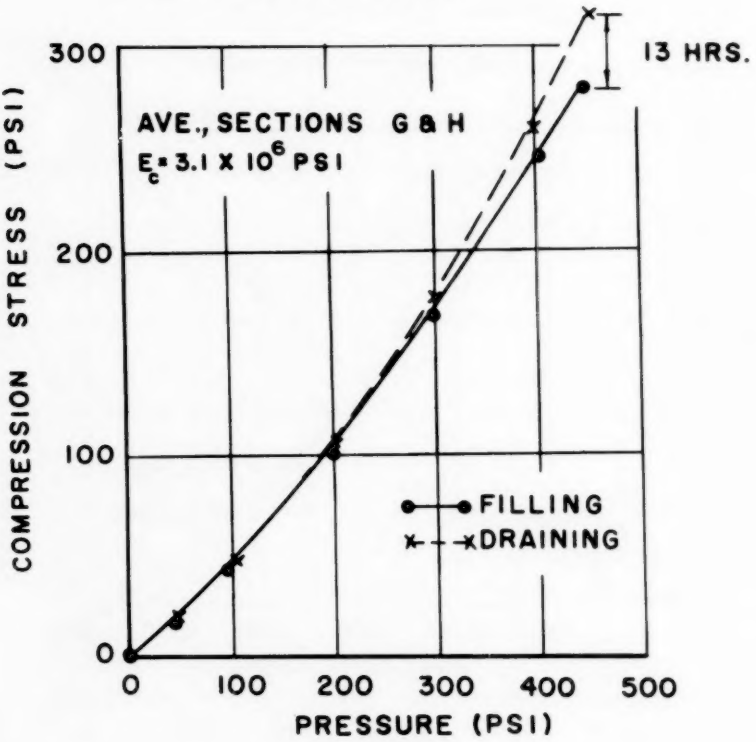


FIGURE II

RADIAL STRESS IN THE CONCRETE OUTSIDE THE 15/16 IN. STEEL AT THE 1680 FT ELEVATION OF PENSTOCK NO. 2

	FILLING	DRAINING
TEMP. NEAR STEEL	52.3° F	51.4° F
TEMP. 3' OUTSIDE STEEL	59.6° F	58.0° F

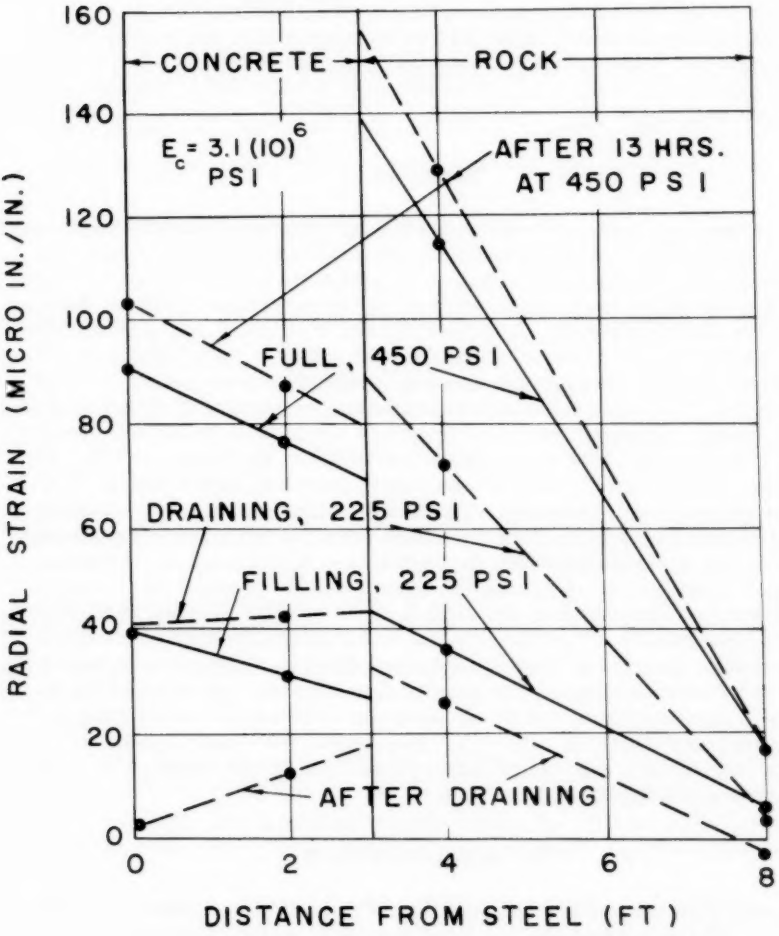


FIGURE 12

RADIAL STRAIN IN THE CONCRETE AND ROCK OUTSIDE THE $^{15}/_{16}$ IN. STEEL AT THE 1680 FT ELEVATION OF KEMANO PENSTOCK NO. 2

at section G at the 1680-ft. elevation is shown in Fig. 12. Some of the gauge readings which were averaged to obtain this graph showed considerable scatter; this was due to variations in rock conditions around this section. During installation of the gauges in the rock and from examination of the drill hole cores it was evident that there were several minor faults in the rock, particularly in one of the upper quadrants. The averaged results indicate that the strain decreases very rapidly in the rock and at a distance of only 10 ft. from the liner it appears to be almost negligible. It is believed that the rock at this section was relatively poor and this is borne out by the relative strains in the rock and concrete at their interface. It is also indicated by the relatively large creep which occurred at this section while the hydrostatic pressure was held at 450 psi.

SUMMARY

In certain instances, a considerable number of advantages may be realized by placing pressure conduits inside a rock mass instead of on the surface. Marked advances have been made in this field since the turn of the century; however, the major design problem, that is, the determination of the distribution of the load between the steel liner and the rock mass, has still not been satisfactorily solved. The principal reason for this is that insufficient fundamental knowledge is available about the elements involved.

In the tests conducted on the first two pressure conduits at Kemano, it has been shown that experimental techniques are available to obtain this fundamental information. The data obtained substantiate, to a certain extent, the applicability of a "cracked-zone" analysis to pressure conduit design, but further information is required. The data also confirm the adequacy of the design of the two conduits which have been installed and provide information which will be directly applicable in the design of future conduits at Kemano.

It was found that for the particular design used in conduits No. 1 and No. 2 at Kemano, the concrete and rock behind the steel liner carried, at most sections, approximately 70% of the load due to the internally applied pressure. Furthermore, quite consistent results were obtained at comparable sections in the two similar conduits. The results also indicate that temperature effects are quite important and should not be ignored and that more information regarding creep effects is desirable. It is hoped that useful information regarding long-term creep will be forthcoming from future readings of the permanently installed gauges at Kemano.

ACKNOWLEDGMENTS

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PENSTOCK EXPERIENCE AND DESIGN PRACTICE[a]

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(Proc. Paper 1397)

SYNOPSIS

The Pacific Gas and Electric Company from long experience in the design of penstocks, has developed certain design practices which it believes to be sound. The paper reviews the recent penstocks designed by the Company and, with this as a basis, presents in detail a summary of the Company's current design procedures.

INTRODUCTION

The design of penstocks continues to be a subject in which improvements are being made. Recent years have seen the development of large amounts of power in single penstocks and this has given incentive to greater refinements in design and application of the latest technological advances. The Pacific Gas and Electric Company has been engaged in an extensive program of expansion of its facilities to meet the rapidly growing needs for power in Northern California. As part of this program, the Company has added a major hydro-electric plant to its system each year, on the average, since the end of World War II. From this accumulated experience, coupled with many years of hydro-electric design and operation, certain practices have developed in the design of pressure pipe.

To summarize these practices, and insofar as possible, show their basis, will be the purpose of this paper. The paper will be confined largely to penstocks constructed or planned since 1948. While the Company has many penstocks that were designed and built prior to this time, it is believed that the basis for the Company's current practices can be established adequately by considering what might be termed the Company's modern penstocks. Only

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steel pipe will be considered inasmuch as all of the Company's modern penstocks have been made of steel with but one exception.

The paper will first review the penstocks designed by the Company since 1948 and will show the changes in the Company's design practice that have taken place in that period. Next will follow a discussion of the design methods in current use including the choice of materials, some comments on the design pressure, the penstock layout, and the design of the pipe shell. This will be followed by some remarks on the details of design such as wyes, elbows, nozzles, and similar items. The paper will conclude with some comments on those phases of the fabrication and erection which influence penstock design.

Review of Company's Modern Penstocks

Table I summarizes data on the penstocks designed by the Company's Engineering Department since the end of World War II. The fourteen penstocks listed represent a considerable range of experience. The total power obtained from falling water in these penstocks is almost a million and a half horsepower and has involved the use of more than 20,000 tons of steel, fabricated into almost 9 miles of pressure pipe. The individual plants range in size from 184,000 horsepower to 3,000 horsepower with static heads up to 2444 ft. Penstock diameters range from 14 ft. 3 in. to 3 ft. In several cases the maximum penstock diameter was set by shipping limitations.

All penstocks designed by the Company have used as a general basis for design the API-ASME Code for Unfired Pressure Vessels. This Code has been used as a guide, but where experience indicated the desirability of doing so, the Company has not hesitated to depart from its provisions. The API-ASME Code, it is understood, will no longer be published, having been supplanted by the ASME Boiler and Pressure Vessel Code, Section VIII, Unfired Pressure Vessels. The general provisions of the two codes are quite similar and, henceforth, the Company will use the ASME Code, as its guide.

All penstocks have been subjected during plant testing and probably at other times, to pressures not less than the design pressure. In at least one case, due to mishap during testing, waterhammer pressures were obtained considerably in excess of design pressure. No damage to the penstock resulted.

Interior surfaces of all penstocks listed in Table I have been coated with centrifugally-spun coal-tar enamel. The Company's experience with this material extends over a period of twenty or more years and has been very satisfactory. Aside from minor abrasions, there have been no failures observed in the coal-tar coating of any penstock. Friction loss in spul coal-tar lined pipes appears to be very favorable. The Company has made a number of tests for determining friction loss in penstocks, but in most instances the head losses in valves, bends, and other minor losses were so large compared to the total loss, that the determination of pipe friction was somewhat in doubt. One exception to this was a test made in 1953 on the Bear River penstock after the plant had been in operation for approximately a year. In this case, a long penstock made accurate knowledge of the minor losses unimportant. Values of Manning's n obtained over a large range of flows were remarkably consistent. The average value obtained was $n = .00971$. The lining in the section of penstock under test was almost entirely machine spun. The value of n for hand-daubed coal-tar enamel is considerably higher.

Choice of Steel

There are a number of steels which, by suitable adjustments in the allowable stress, could be used for penstock construction. The ASME Code affords a wide selection and, in addition, there are several steels now on the market having very high yield points which could be considered. Some aid in the final choice of a steel for a penstock can be obtained by cost estimates prepared for comparative designs of penstocks having equal safety. However, the preparation of equal safety designs, where not only the steel, but fabrication methods, and other variables are introduced, becomes very difficult and the final selection rests largely on the experience and judgment of the designer.

In general, the modern penstocks on the Pacific Gas and Electric Company's system are shop welded, x-rayed and stress relieved, and joined in the field by riveting. Under these circumstances, it is believed that weldability is a very important requirement of a penstock steel. In general, the Company does not permit field welding of penstock joints, nor will it permit any welding on fabricated pieces of pipe which have been previously x-rayed and stress relieved.

Within these limitations, the problem of choosing the steel to give the most economical pipe has been studied by the Company a number of times. In almost all cases, the steel chosen has been A.S.T.M. A 212 Grade B, Firebox Quality. To weld this steel successfully requires pre-heating and careful shop control, but the results with the steel have been entirely satisfactory.

While it is generally true that the Company has chosen A 212 steel for penstocks, inspection of Table I reveals a number of exceptions. In several penstocks (Colgate, Rock Creek, Murphys, Alta) near the upper end where the thickness of plate is determined by an arbitrary minimum and not by stress, the steel used has been A 285. Alternate bids were taken on the Cresta penstock in A 212 and A 285. The pipe was constructed of A 212. The Company does not favor field welding of penstocks; however there are conditions where it is manifestly impracticable to do otherwise, such as the large diameter tunnel liners connected to penstocks at the upper end and solidly encased in concrete. In such cases (Rock Creek, Butt Valley, Caribou No. 2, for example) A 285 has been specified. In the case of the Haas project, the lower portion of the penstock in the shaft was designed for A 242 to be field welded.

Design Pressure and Allowable Stress

The problem of selecting a design pressure and the choice of a corresponding allowable stress are considered together. It is believed that this is desirable, inasmuch as the pressure taken for design purposes is a hypothetical one that is probably never duplicated exactly in the plant when built. This being the case, the designer must choose an appropriate allowable stress, taking into account the severity of the design load and the porbability that it might be exceeded.

Figure 1 illustrates the three types of pressure assumptions used for the design of the Company's recent penstocks. The type load used for individual penstocks and the corresponding allowable stress are given in Table 1.

TABLE 1
RECENT PENSTOCKS
PACIFIC GAS AND ELECTRIC COMPANY

Plant	Completion Date	Design Flow cfs	Plant Capacity h.p. (1)	Static Head Ft.	General Description
Electra Mokelumne R. Amador Co.	1948	975	112,500	1268	Single steel pipe supported on piers dividing to 3 and then 6 pipes at P.H. Three double-overhung impulse units with jet deflectors.
West Point Mokelumne R. Amador Co.	1948	575	17,000	312	Single steel pipe upper half supported on piers, remainder buried. Reaction turbine with relief valve.
Colgate N.F. Yuba R. Yuba Co.	1949	470	35,000	820	Single steel pipe supported on piers. Reaction-turbine with relief valve.
Rock Creek N.F. Feather R. Plumas Co.	1950	3000	147,000	535	Two steel pipes: Upper half supported on piers; lower half in concrete-backfilled shaft. Reaction-turbines with relief valves.
Cresta N.F. Feather R. Butte Co.	1950	3500	93,000	290	Two steel pipes supported on piers. Reaction-turbines with relief valves.
Bear River Mokelumne R. Amador Co.	1952	200	53,500	2109	Single steel pipe supported on piers, except short length at bottom in concrete-backfilled shaft. Single 3-nozzle impulse unit with jet deflectors.
Murphys Stanislaus R. Calaveras Co.	1953	80	5,300	684	Single buried pipe: Upper third wood staves; remainder steel. Single 6-nozzle impulse unit with jet deflectors.
Pit 4 Pit R Shasta Co.	1955	3500	124,000	382	Two steel pipes supported on piers. Reaction-turbines with relief valves.
Alta (?) Little Bear R. Placer Co.	1955	50	3000	648	Single buried steel pipe dividing to two pipes at P.H. Two single overhung impulse wheels with jet deflectors.
Poe N.F. Feather R. Butte Co.	1958	3500	146,000	477	Single steel pipe, in concrete-backfilled tunnel dividing to two pipes as it emerges from tunnel and enters P.H. Two reaction turbines with relief valves.

Pipe Sizes	Approx. Total Length Ft.	Approx. Tonnage Steel	ASTM Type Steel (2)	Maximum Allowable Stress psi (13)	Type Design Load (5)	Joints Between Sections
Single: 120" x 7/16 to 72" x 1½" Three pipes: 52" x 1" Six pipes: 36" x 3/4"	4035	2300	A212B (10)	15000	A	Std. (6)
120" x 7/16 to 84" x 3/4	623	151	A285B (10)	10000	A	Std. (6)
96" x 5/16 to 66" x 18	1633	400	A212B (3)(10)	13000	A	Std. (6)
144" x ½ to 126" x 1-3/16	1860	1290	A212B (3)(10)	13000	A	Std. (6)
144" x ½ to 144" x 15/16	1579	869	A212B (10)	13000	A	Std. (6)
72" x 3/8 to 36" x 1-11/16	4698	1615	A212B (10)	13500	A	Std. (6)
Wood stave: 48" d. Steel: 47" x ¼ to 24" x 7/16	Wood: 1657 Steel: 2395	175	A212B (3)(10)	13500	A	Dresser couplings
144" x ½ 144" x 1	1593	1086	A212B (10)	15000	C	Std. (6)
36" x ¼ to 36" x 3/8	5383	286	A212B (3)(11)	14875	B	Dresser couplings
Single pipe: 171" x 15/16 to 171" x 1½ Two pipes: 120" x 1	985	1100	A212B (10)	15000	B	Std. (6)

TABLE I continued

Plant	Completion Date	Design Flow cfs	Plant Capacity h.p. (1)	Static Head Ft.	General Description
Butt Valley					
Butt Creek Plumas Co.	1958	1700	55,000	368	Single steel pipe supported with ring girders on piers. Reaction-turbine with relief valve.
Caribou No. 2					
N.F. Feather R. Plumas Co.	1958	1350	150,000	1150	Single steel pipe supported on piers dividing to two pipes on entry to P.H. Two 6-nozzle impulse units with jet deflectors.
Haas					
N.F. Kings R. Fresno Co.	1958	760	184,000	2444	Single steel pipe supported on piers except lower one-sixth in concrete-backfilled shaft. Pipe divides on entry to underground P.H. Two 6-nozzle impulse units with jet deflectors.
Balch Addition					
N.F. Kings R. Fresno Co.	1958	560	134,000	2390	Single steel pipe supported on piers dividing into two pipes on entry to P.H. Two 6-nozzle impulse units with jet deflectors.
Belden (4)					
N.F. Feather R. Plumas Co.	1960	2250	156,000	770	Siphon and penstock: Single steel pipe supported on piers. Penstock divides on entry to P.H. reaction turbines.
TOTALS			1,415,300		

(1) Rated capacity of prime movers.
(2) Steel indicated is that used where thickness is determined by stress.
(3) A285B steel is used at the upper end where pipe thickness is set by minimum requirement.
(4) Design is not complete at the time of writing. Data on preliminary.
(5) See Fig. 1 for explanation.
(6) Refers to type shown in Fig. 2
(7) Penstock was a replacement of one that had depreciated beyond possibility of repair.
(8) 29000 if steel assumed to carry entire load; 19000 if load is assumed to be shared with surrounding rock.

TABLE I continued

Pipe Sizes	Approx. Total Length Ft.	Approx. Tonnage Steel	ASTM Type Steel (2)	Maximum Allowable Stress psi (13)	Type Design Load (5)	Joints Between Sections
138" x 7/16 to 132" x 11/16	5560	2140	A212B (10)	17500	B	Std. (6)
Single pipe: 126" x 9/16 to 102" x 1-11/16 Two pipes: 72" x 1-3/16	2316	1380	A212B (10)	17500	C	Std. (6)
Single pipe: 96" x 9/16 to 76" x 2-11/16 Two pipes: 82" x 2-1/4	4555	3088	Above ground: A212B (10) In shaft: A242 (9)	A212B: 17500 A242: (8)	C	Std. (6) above ground; field butt weld below
Single pipe: 96" x 3/8 to 68" x 2-3/8 Two pipes: 46" x 1-5/8	4980	2735	A212B (10)	17500	C	Std. (6)
Siphon: 156" x 9/16 to 144" x 1-1/8 (Penstock: (Single pipe: 156" x 9/16 to 138" x 1-3/8 Two pipes: 98" x 1	Siphon: 2100 Penstock: 1420	Siphon: 1270 Penstock: 1000	A212B (10)	17500	B	Std. (6)
	45715(12)	20885				

(9) Shop welds are x-rayed and stress-relieved; field joints are x-rayed and field stress-relieved at extreme lower end.
(10) X-rayed and stress-relieved.
(11) Stress-relieved but not x-rayed.
(12) Includes steel pipe only.
(13) 100 percent weld efficiency assumed.

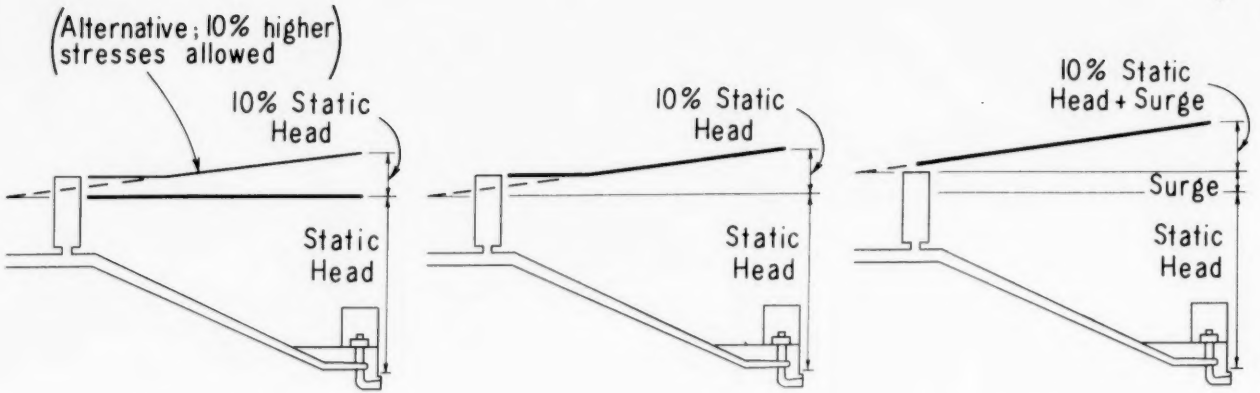

FIGURE I
DESIGN PRESSURES ON PENSTOCKS

Notes: (1) Design pressure at any point is indicated by the heavy line.
(2) Base of the water hammer triangle is the developed length of the water conduit.
(3) Static level corresponds to maximum reservoir level.

Design Pressure

Consideration of the Type A load, (Fig. 1) which was used by the Company until about 1953, will indicate that the two alternate load lines shown would yield exactly the same penstock only when the penstock profile lies on the straight line between the surge tank water surface and tailwater. Inasmuch as most penstock profiles lie below this line, it is apparent that the static pressure line would almost always be the governing condition. Such was the case in the seven penstocks shown in Table 1 that were designed on the basis of the Type A load.

Beginning with the design of the Pit 4 penstock about 1953, the Company adopted either the Type B or Type C load. It is believed that either of these two loads gives a better approximation than Type A load to the actual loading condition on penstocks and hence yields a more balanced design. The choice between the Type B and Type C load for an individual plant should be based on a study of the particular situation. Among the factors which affect this decision are (1) the speed of governor operation, (2) estimated probabilities of accidental high waterhammer pressures, (3) speed of water level changes in the surge tank, (4) the consequences of penstock failure, and (5) the cost of providing the additional margin of safety afforded by the Type C load.

The waterhammer allowance of 10 percent static shown in Fig. 1 is empirical, but has been adopted because in the Company's experience, this allowance for waterhammer has proven to be adequate. It is impossible to compute the magnitude of waterhammer pressures unless the rates of shut-off and other factors are known. Under the design and construction schedule generally imposed on the Company's projects, the penstock must be designed long before the characteristics of turbines and shut-off valves are available. As a result, an arbitrary allowance of 10 percent is made for waterhammer and, when the plant is built and tested, the governor and relief valves are so adjusted that the pressure rise, which is easily measured at this time, does not exceed the allowance made in design. Among the recent plants built by the Company, there have been no instances where waterhammer pressures could not be restricted to the 10 percent limit.

There are situations where a 10 percent allowance for waterhammer would not be adequate. A designer might conclude, for example, that a penstock should be safe if one jet of a multiple-nozzle impulse turbine should close instantaneously. Such a design condition would add relatively more material to the upper end of the penstock than the lower, inasmuch as an instantaneous waterhammer wave moves up the penstock undiminished.

Another condition where allowance for higher waterhammer pressures should be made, occurs where the relief valve associated with a reaction turbine is not used or where it has less capacity than the turbine. In this case, not only higher waterhammer pressures must be accepted, but higher overspeed of the unit results, and the problem becomes one of balancing the saving resulting from the elimination of the relief valve, against the increased cost of the penstock, combined with the increased cost of the turbine-generator.

Allowable Stress

Current practice of the Pacific Gas and Electric Company is to design penstocks using an allowable stress for A 212 B steel of 17500 pounds per square inch with 100 percent weld efficiency assumed. This is applied to steel which has been x-rayed and stress relieved in the shop, and is used with either

Type B or Type C design load. To bring all the allowable stresses shown on Table I approximately into a comparable basis, the stresses used with the Type A load should be increased by 10 percent. If this is done, it will then be noted that the Company has been steadily increasing allowable penstock stresses. This reflects increasing confidence in the design methods used by the Company, the steels available, and the fabrication processes employed.

Combined stresses in the penstock plate are minimized as much as possible. Bending stresses are inevitable in a surface penstock and other minor stresses accumulate in the longitudinal direction from friction in the pipe supports and the weight of a sloping pipe and other loads. It is believed an ideal penstock would be one without stress of major magnitude in the longitudinal direction. Where such stresses are unavoidable, the Company's practice has been to limit the combined stresses as required by one of the accepted theories of failure.

Penstock Layout and Design of Shell

A number of formulas have been proposed for the computation of the required thickness of the pipe shell. The Company favors the use of the Code formula:

$$h = \frac{4.6\ st}{d + 1.2t}$$

when h = maximum allowable head on the pipe centerline in feet.

s = allowable hoop stress, lbs. per sq. in.

d = inside diameter in inches.

t = thickness of pipe shell, inches.

Because it is slightly more convenient, the Barlow formula has been commonly used in the Company's designs. This takes the form:

$$h = \frac{4.6\ st}{d + 2t}$$

where the symbols have the same meaning as before. The difference between these two formulas is negligible except for thick pipes. No allowance is made for corrosion because the pipe is given a protective coating inside and out.

The problem of selecting the diameter of a penstock is an economic one. The most economic penstock is the one in which the annual cost of the penstock, combined with the value of the power lost annually be penstock friction, is a minimum. By consideration of a length of penstock one foot long, an expression can be derived which gives the head at which a penstock of a particular diameter has maximum economy. The equation used by the Pacific Gas and Electric Company takes the form:

$$h = \frac{0.0372\ Pn^2Q^2S}{Ci} \cdot \frac{1}{d^{22/3}}$$

where h = head in feet where annual cost is minimum.

P = annual power value of one foot of head in penstock.

n = Manning's n

Q = design flow, cubic feet per second.

S = design stress on penstock, pounds per square inch.

i = factor to convert first cost to equivalent annual cost.

C = first cost of penstock in place, dollars per pound.

d = diameter of pipe in feet.

As a practical matter, the results obtained by using this formula are modified so that changes in diameter occur at convenient places in the penstock. Ordinarily, changes in diameter are made at the bends in the pipe. This is done to reduce the number of specially detailed pieces of pipe. Where changes in pipe diameter are made in straight sections of pipe, the taper section is made about a diameter in length and is ordinarily incorporated in a full-length section of pipe.

Pipe sections on the Company's penstocks are ordinarily detailed 40 feet long. In cases where the pipe has been very thick or of very large diameter, it has occasionally be necessary to restrict the length to 30 feet in order to keep the sections within manageable weight. Length of penstock sections is the same as the distance between piers.

Very thin-walled pipes without stiffening rings tend to sag badly out of shape due to their own weight when there is no internal pressure. In order to avoid this, it is necessary to adopt some arbitrary minimum thickness. Minimum plate thickness used by the Pacific Gas and Electric Company for plain pipe is:

$$t = \frac{d}{288}$$

This results, for example, in half-inch plate for twelve foot pipe.

Joints Between Pipe Sections

Most of the penstocks designed by the Pacific Gas and Electric Company since World War II have used the field joint shown in Fig. 2. The joint was developed as a result of the Company's belief that welding, if done in the shop, radiographed, and stress relieved, is satisfactory, but that field welding is not sufficiently reliable to justify its use in penstocks.

Joints in surface penstocks are ordinarily placed at the lower quarter-point of each span. This procedure facilitates erection of the penstock and places the joint near the point of inflection in the span. Joints are designed for whatever longitudinal load can be computed to exist in the pipe, but in no case is the joint efficiency less than 25 percent. The thickness of butt strap is ordinarily not less than two-thirds of the shell thickness and the material is the same as the shell. Allowable stresses for design of the joint are: tension on net section same as allowable stress in pipe shell; rivet shear 11,000 lbs. per sq. in., bearing 23,750 lbs. per sq. in. Rivet holes are assumed to be 1/16 in. larger than the nominal rivet diameter. Rivet spacing in the outer row of rivets is limited to an arbitrary maximum in order that the edge of the buttstrap may be effectively calked. It is preferred that the rivets be not calked, and in most cases, it is unnecessary provided the rivets are driven hard enough to completely re-form the head to the steeple profile. The rivets are ordered 1/4 to 3/8 inch longer than the standard lengths for

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FIGURE 2
STANDARD JOINT BETWEEN SECTIONS

countersunk heads. This results in a little excess metal in the oval-countersunk head and makes it possible for the full blow of the riveting gun to fall on the rivet instead of the plate around the periphery of the head. In the case of the Electra penstock, the rivets were bucked-up from the inside and driven outside. Penstocks built since then have utilized what seems to be a better practice and have been driven from the inside. This type joint has been used on pipe shells up to 1-3/4 inches in thickness. Maximum size rivets that can be effectively driven in the field is 1-1/4 inches.

Whenever rigid joints are used to join pipe sections, it is usually necessary to have an expansion joint between each pair of penstock anchors. An expansion joint used by the Pacific Gas and Electric Company is shown on Fig. 3. The entire assembly is fabricated by welding methods comparable to the methods employed in making pipe shells. In the past, nickel-clad plate has been specified with the cladding about 1/16 inch thick. The clad plate was rolled into a cylinder and the longitudinal weld, when completed was overlain by a welding pass made with a nickel-bearing rod. More recently, it has been impossible to obtain nickel-clad steel and it has been necessary to accept stainless clad. It is believed this will prove to be satisfactory, but the material has yet to be put in service. The extent of motion of the expansion joint is set by the requirements of field erection. Ordinarily, the expansion joint is placed at the upper extremity of the run between anchors and the section of pipe containing the expansion joint is the last section to be placed. This section, with the expansion joint fully collapsed, must be capable of rolling into the remaining space and then being expanded to make up the joints at either end. The packing material used is square, plaited from asbestos yarn graphited throughout. This packing material has proven to be very satisfactory.

On buried penstock, no expansion joints are used. It is believed that the temperature range to which a buried pipe is subjected is not large enough to make their use necessary.

The flexible type of penstock coupling, such as the Dresser, offers some advantages over a rigid type connection. The requirement for expansion joints is eliminated and in many cases, by burying the pipe in long gradual curves, the number of elbows and anchors can be greatly reduced or eliminated entirely.

Among the Company's penstocks, only the Murphys and Alta penstocks are completely assembled with flexible type couplings. Elsewhere flexible type couplings have been used to provide a section of pipe which can easily be removed from the line. In this case, the section is connected into the line with flexible couplings from which the pipe stops have been eliminated. To remove the pipe section, the packing nuts are loosened and the couplings are slipped along the pipe.

Elbows and Wyes

Inasmuch as elbows, together with their required anchors, are considerably more costly than straight sections of pipe, their use should be minimized in the layout of the penstock. Horizontal and vertical angles are usually made to occur in the penstock at the same point and one elbow is made to the combined angle.

Elbows, in the Pacific Gas and Electric Company's practice, are usually

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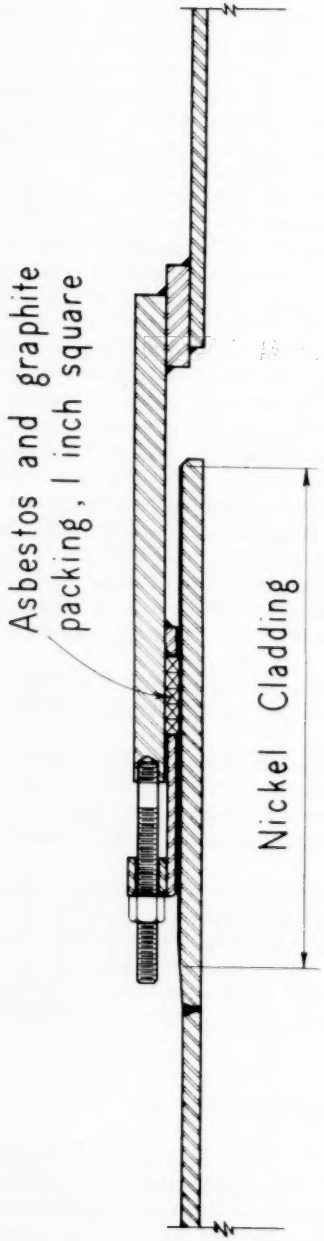


FIGURE 3
EXPANSION JOINT

made with a centerline radius of four pipe diameters. The total deflection angle to be turned is made in a series of mitre joints between cylindrical or conical sections of pipe. Deflection angle at the mitres is limited to 6 or 7 degrees, if possible, though occasionally slightly larger deflections are made necessary to give a practical elbow. The shell thickness of elbows is computed by the same formula as used for straight pipe. Very large forces must generally be taken out of the pipe at elbows and it is necessary to use thrust rings and hold-down straps for this purpose. If an elbow also serves as a taper section, the location of thrust rings and hold-down straps so as to lie flat against the pipe shell is sometimes troublesome. The design of thrust rings and hold-down straps will be discussed below in connection with anchors.

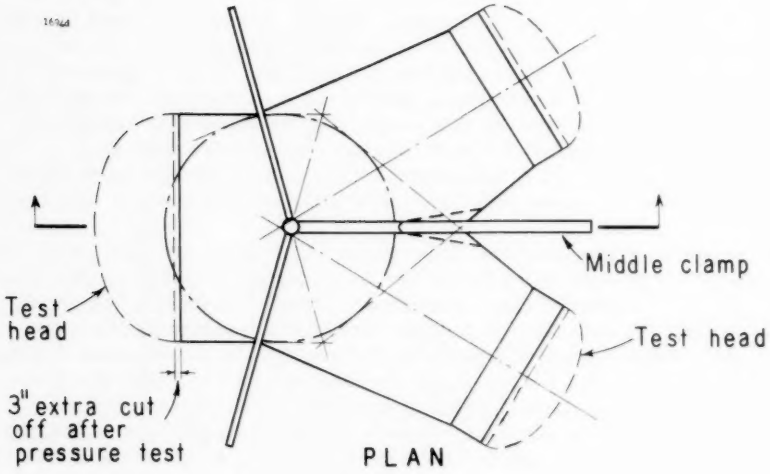
Wyes are used in a penstock layout either at the extreme lower end where the penstock is divided into a branch for each unit of the power house or they may be required where studies of economic diameter show that two pipes, each with half the total flow, is a more economical arrangement than the single line. Wyes designed recently by the Pacific Gas and Electric Company have all been of welded design except for Electra which has cast-steel wyes. The welds are radiographed insofar as possible and the wyes are stress-relieved. All wyes are pressure tested in the shop to a pressure one-and-one-half times the design pressure. Fig. 4, showing the method of layout, is typical of the Company's recent designs. Design stress in the shell of the wyes has been taken the same as the allowable stress in the penstock shell. Somewhat lower allowable stresses have been used in the clamps because it was believed that curved beams are less capable of exact analysis than hoop stress in a pipe. As much as possible of the middle clamp is placed outside the wye, but in some cases the maximum shipping dimensions have required that much of the middle clamp be inside where it becomes more nearly a center diaphragm. In the case of the Haas plant, two alternate designs, one of the spherical type and the other similar to Fig. 4, were prepared for a design pressure of 1172 pounds per square inch. Subsequent bids showed the spherical type to be the more expensive.

Other Steel Details

Manholes and Nozzles

In the case of manholes and nozzles, the practice of the Company has been generally to follow the recommendations of the Code, though in most instances, designs have been made on a more conservative basis than the Code requires. A typical nozzle is shown in Fig. 5a. Under the Code, the designer is required to replace 100 percent of the area removed from the pipe shell and certain parts of the nozzle itself as well as the reinforcing plate and the excess thickness in the pipe shell can be used as replacement metal. The Company's practice in this respect is to replace 125 percent of the removed area and to assume that only the reinforcing plate is effective in making the replacement. While this practice is a little more conservative than the Code, it is believed that the extra safety provided by these relatively minor additions is justified.

Manholes are designed by the same principles as any other pipe nozzles. For a number of years, it was the Company's practice to use a 16-inch clear opening, but it has been discovered more recently that our employees are



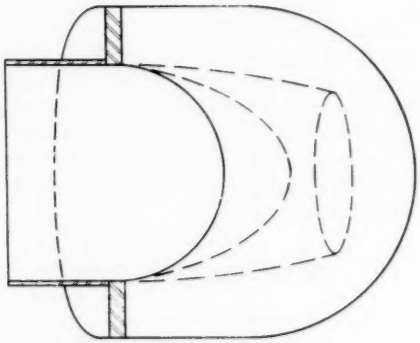


FIGURE 4
TYPICAL WYE BRANCH

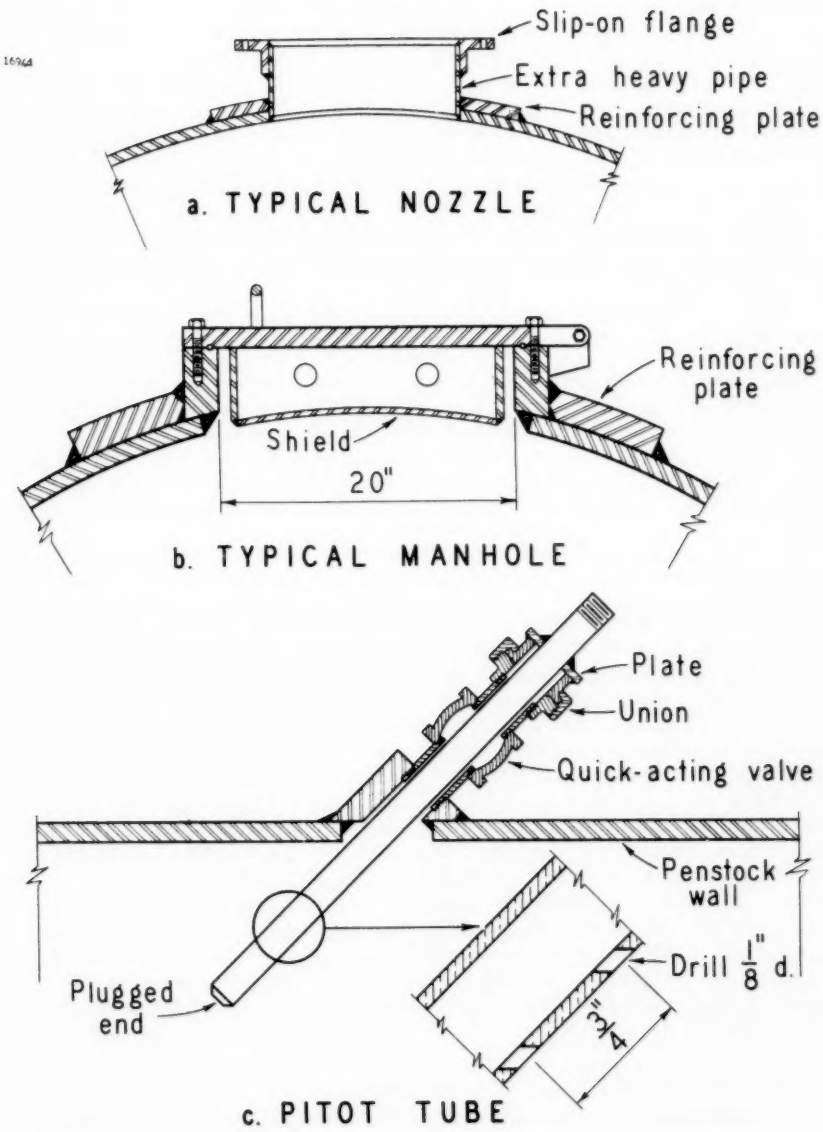


FIGURE 5
MISCELLANEOUS DETAILS

better adapted to 20-inch manholes and this size is now used wherever practicable. A typical manhole is shown in Fig. 5b. On our most recent penstocks we have adopted the practice of adding a shield constructed of light metal for the purpose of smoothing the interior of the penstock and avoiding the turbulence created by the depression in the pipe wall. Manhole covers and bolts are designed by the Code with round rubber gaskets used for sealing. Manholes are located in the cross-section of the pipe on the bottom half, 45 degrees off the vertical. This location generally makes it possible to gain admittance to the pipe without the use of a ladder on the inside. Manhole covers are provided with hinges for easy handling. Manholes are ordinarily provided at the upper and lower ends of penstocks and at intervals of about 500 feet between.

Roll-out Sections

At the lower end of long tunnels where access to the tunnel is not available in some other way, the Company's practice has been to provide a section of penstock that can be easily separated from the line and rolled out. The roll-out section of pipe is connected to the adjacent pipe with Dresser-type couplings. The removable section, which is generally 10 or 12 feet in length, sometimes has wheels attached, and is rolled aside on permanently installed rails.

Valve Trips

At the upper end of penstocks, common practice is to provide a valve for the purpose of shutting off the water if the penstock should break. This necessitates employing some device which senses an excess flow of water and trips the valve. While there are other ways of performing this function, the Company has, in all cases, used pitot tubes for the sensing device. These have not proven to be completely satisfactory for reasons which apply as well to other systems as to pitot tubes, (1) the tubes can become clogged with floating moss and other debris, and (2) a partial break in the penstock in which the flow could be quite large might go undetected, particularly if the power house were not operating at the time. The pitot tubes now used by Pacific Gas and Electric Company are shown diagramatically in Fig. 5(c). The arrangement makes it possible to withdraw the tubes easily for cleaning. The tubes are slanted downstream in the flow in order that floating material will be swept on, instead of clogging the tubes. Experience has shown this to be reasonably effective. Difference in head between the slanting tubes pointing upstream and those pointing downstream has proven to be about 1.3 times the velocity head in the penstock.

Standpipes and Air Valves

Just downstream from the valve at the head of a penstock it is necessary to provide a connection to the atmosphere. The connection serves two purposes: (1) in case the upper penstock valve closes, air must be admitted to avoid collapsing the pipe from external pressure, and (2) in filling through the by-pass, it is necessary to allow the air in the penstock to escape. These functions are performed by either a standpipe or an air valve. The standpipe is probably the more reliable, but in the experience of the Pacific Gas and Electric Company, air valves have proven to be very much cheaper.

The principles of design of both standpipes and air valves are well known. In the past, the Company has used air valves up to 12 inches in diameter with cast steel bodies, but more recently there has been devised an air valve of welded design which seems to have a number of advantages, including cost, over the cast-body design. Sketches of the two types are shown in Fig. 6. In both cases, the moving element is supported by the spring in such a way that the valve is open while air is escaping from the penstock, but closes as soon as water attempts to pass. The valve seats are made of bronze lapped to an air-tight fit. The central shaft is of stainless steel running on bronze bearings. A fitting is provided in the body of the valve for oil injection. Oil injected into the water trapped within the valve body rises and forms a layer next to the seat. This precaution has proven to be sufficient to prevent freezing of the valve. The oil is lost and must be replaced whenever the valve operates, but operations of the valve are quite infrequent. Other methods of preventing freezing are to house or to insulate the valve. For purposes of reliability of operation, it has been the Company's practice to use the valves in clusters. A minimum of two valves is always used and more are employed to obtain the requisite air capacity.

Anchors and Piers

The general principles for design of penstock anchors is well known and will not be discussed here. There are a number of details of anchors, however, where the experience of the Pacific Gas and Electric Company might be interesting.

Anchors are placed at every penstock bend and at intermediate points to limit the maximum penstock length between anchors to about 500 feet. A typical anchor is diagrammed in Fig. 7. If possible, no field joints are included in the length of pipe within the anchor. The thrust rings are made of channels rolled to the curvature of the pipe and welded to the pipe shell in the shop before stress relieving. Thrust rings are designed to withstand the axial stress in the pipe and transfer it to the anchor. They are located near the ends of the anchor so that the pipe in the region of the elbow will be relieved of longitudinal stress. Hold-down straps are made of flat bars of sufficient size to withstand the force tending to raise the pipe from the concrete. At least two hold-down straps are used on all anchors. Each flat bar is held in close contact with the pipe shell by a pair of threaded rods passing under the pipe. At the lower end of the hold-down straps, two short lengths of channel are bolted which are proportioned to develop the strength of the hold-down straps by embedment in the concrete. Hold-down straps are shipped separately and attached to the pipe in the field while it is being erected. The concrete portion of the anchor is separated into two pours. The first pour furnishes a base on which the pipe can be erected and is suitably keyed on its upper surface to engage the second pour. The entire first pour can be completed without embedding any part of the pipe or its hold-down straps. The keys are arranged on the first pour so that when the pipe is placed, the shoes on the hold-down straps fit into the keying depressions with ample clearance. Reinforcing bars placed in the first pour near the hold-down straps can, if desired, be welded to the shoes of the hold-down straps to assist in holding the pipe in place for the second concrete pour. The second pour, which is brought up to the horizontal diameter of the pipe, completes the anchor.

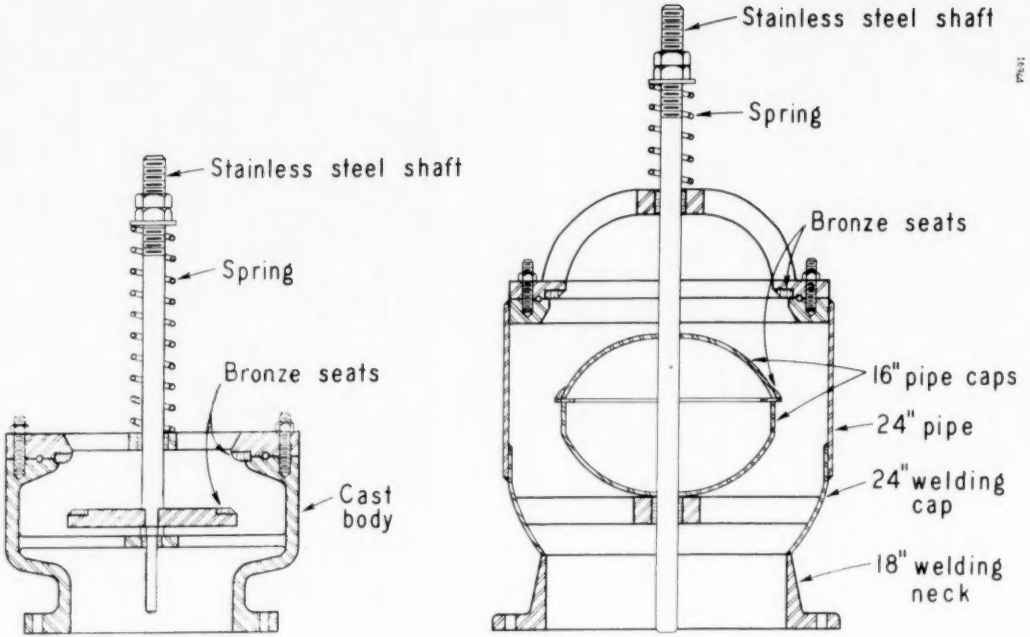


FIGURE 6
AIR VALVES

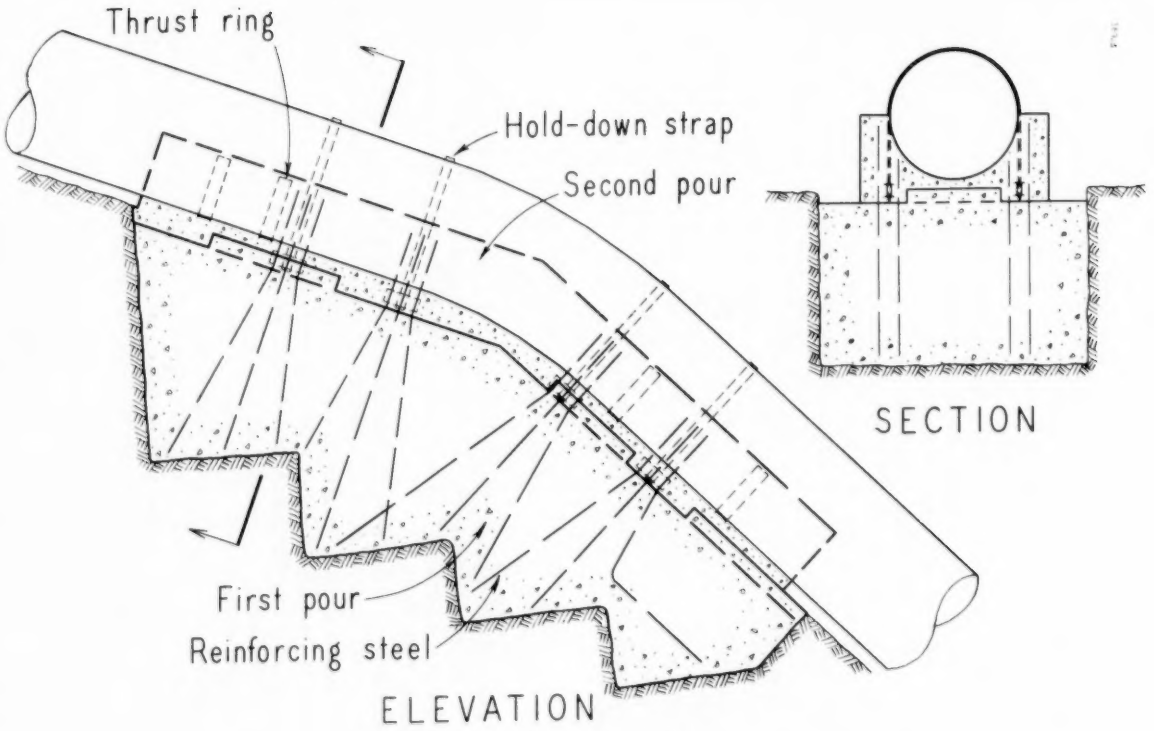


FIGURE 7
TYPICAL PENSTOCK ANCHOR

Intermediate piers between anchors for the purpose of supporting the weight of the pipe have been constructed in several forms. A type commonly used by the Pacific Gas and Electric Company is shown in Fig. 8a. This type of pier, though no rational stress analysis of the adjacent penstock seems possible, has proven very satisfactory even on pipes 12 feet in diameter and half-inch thick. The concrete in the pier is reinforced and the pier is computed to be stable for a coefficient of friction of 0.5. The first pour of the pier is the bottom slab which provides a convenient support for cribbing to support the pipe during erection. The packing between the steel and concrete is made up of two sheets of bonded-asbestos sheet packing with their graphited faces together. The packing is wired to the penstock with a mastic adhesive between packing and steel. The last step in construction of the pier is to round the pipe carefully and pour the concrete in the stem.

The type of rocking support shown in Fig. 8b is used where it is necessary to gain height in the pier. With suitable bracing for lateral strength this type of rocker can be built to almost any height. There is no machine work involved and all parts are welded and stress relieved in the pipe shop.

The detail shown in Fig. 8c proved to be an economical arrangement on the Butt Valley penstock where the spans were long (54 ft.) and the grade was relatively flat. At Butt Valley the pipe is supported on ring girders with the pipe shell somewhat thickened near the rings. The bronze bearing is permanently impregnated with lubricant. The reduced friction obtained by the use of the bronze bearing results in a smaller pier than would otherwise have been possible.

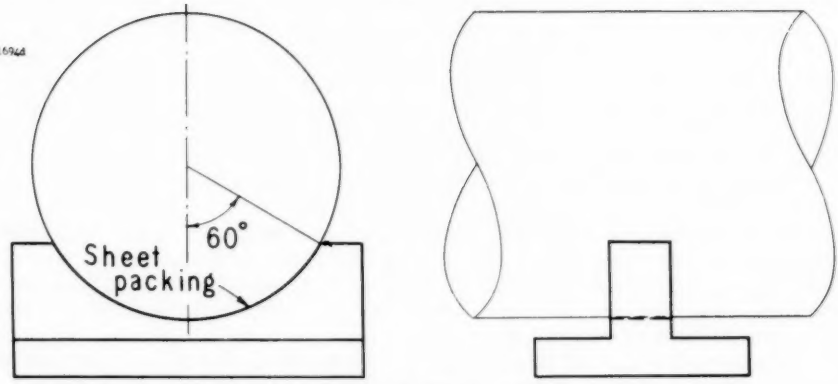
Fabrication

It is not the purpose of this paper to discuss the fabrication of penstocks except insofar as shop techniques affect penstock design. What follows therefore, merely touches on some of the highlights of shop practice where these practices seem significant.

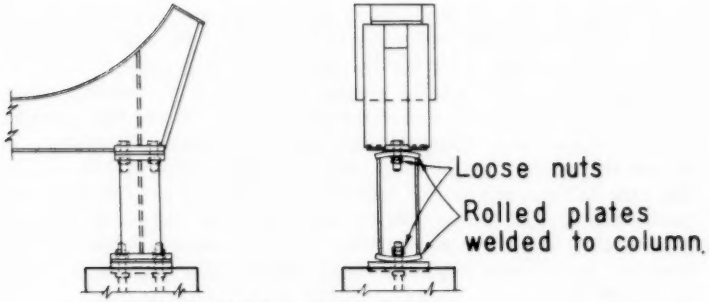
Not all pipe shops adopt the same methods and their practices are largely dictated by available tools. For the ordinary thicknesses, practices are fairly uniform. The edges of the plates to be joined are prepared in a planer where the dimensions of the plate are accurately laid out. It is found that after these plates are crimped along the edges, rolled to a cylinder and tack welded, that the diameter is remarkably close to the design diameter. To maintain tolerances of 1/16 inch, even on very large pipe, is not unusual. This is of considerable importance in the fit-up of joints.

The plates are made into cans, five to ten feet long depending on the width of plate used. All welding, except tack welding, is done with automatic welding machines. Reinforcement of the welds is not ground off except where it would interfere with assembly of the pipe.

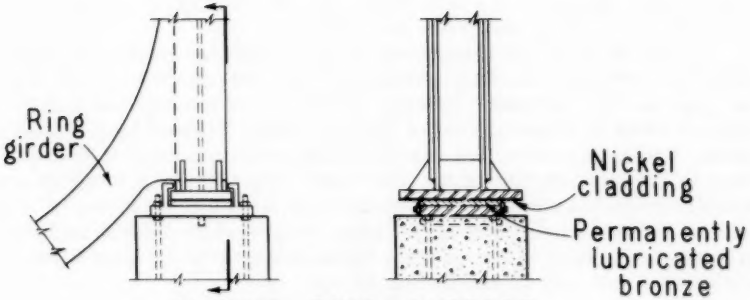
The joint shown in Fig. 2 is fitted-up in the shop using adjoining cans before the pipe sections are assembled. The butt strap is cut to a measured length, rolled into a hoop and welded. So accurately is the butt strap made that when the butt strap is pressed onto the can in a jacking press, metal-to-metal contact throughout is usually obtained. When the butt strap is welded to the can, it is sometimes found to have belled-out slightly. This is corrected by shrinking the butt strap (heating and quenching) until metal-to-metal contact is obtained in the riveted part of the joint. With the two cans together,



a. CONCRETE PIER



b. WELDED ROCKER



c. SLIDING RING SUPPORT

FIGURE 8
PENSTOCK SUPPORTS

the rivet holes are reamed to nominal rivet diameter and countersunk.

A pipe section is completed by making the round-about welds and adding any special items such as manholes and nozzles. It is then radiographed completely, chipped and re-welded where necessary, and then stress-relieved. The Company does not permit partial or localized stress-relieving. There are several fabricators in the San Francisco area whose stress-relieving furnaces are adequate for the largest pipe the Company has required.

Painting of the penstock interior is done in the shop with hot coal-tar enamel. Plain sections of pipe can be spun and the coal tar enamel is deposited in the spinning pipe. This results in a smooth glassy finish. Sections of pipe which cannot be spun, such as elbows or sections with manholes are hand-daubed. The hand-daubed coating seems to protect the steel as well as the spun lining, but it is quite rough and is to be avoided wherever possible to minimize hydraulic friction losses.

The Company's present practice is to have one shop coat of red-lead primer applied to the outside of the pipe before the pipe is shipped to the field.

Erection

Penstocks are generally erected beginning at the anchors and working uphill. The elbows are set in their anchors and the straight sections of pipe are laid between them. Ordinarily, the last piece of pipe in a run is the uppermost piece and contains the expansion joint. With the expansion joint fully collapsed, the last piece is short enough to roll into the remaining space.

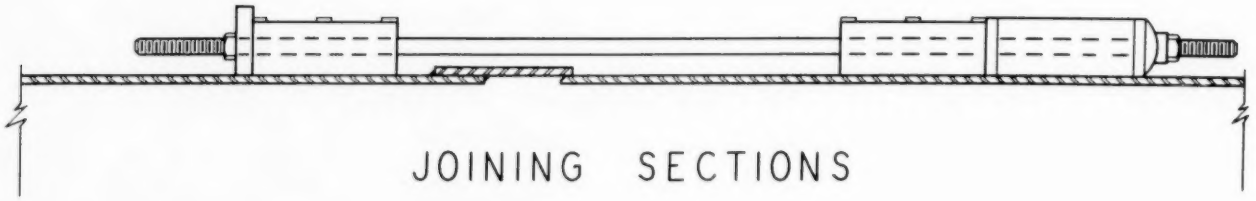
The riveted joints are put together in the field by jacking. The method is shown in Fig. 9. The jacking clips are made of light metal and are welded to the pipe in the shop before stress relieving. Generally four such clips have been used around the periphery of a pipe joint, but on the 14 ft. 3 in. diameter pipe at Poe, six jacks were used. By this method, it has been possible to put together in the field any joint that could be pulled apart in the shop regardless how tight the fit.

Once the joints are brought together satisfactorily, they are bolted with at least half the holes being filled. This permits the erection crew to advance without waiting for the riveting crew. Before riveting, the holes are reamed to 1/16 inch over nominal rivet diameter.

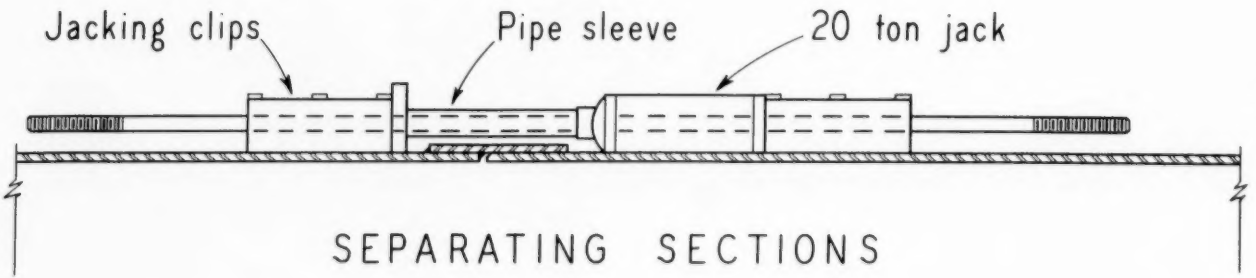
The inside of the pipe, including the area near the joints and any spots that have been abraded, are hand painted with hot coal-tar enamel before closing the pipe for the last time. Exterior protection of the exposed portion of the pipe consists of a second coat of red lead paint, followed by a coat of aluminum paint. The highly reflective surface of the aluminum paint is believed necessary to keep the interior of the pipe, when empty, below a temperature that would damage the coal-tar enamel, and also to minimize motion of expansion joints. Outside surfaces of buried pipe are generally painted with coal-tar enamel and, to keep this coat from being damaged by the backfilling operation, are given a thin coat of reinforced gunite.

ACKNOWLEDGMENT

The penstock practices used at the present time by the Pacific Gas and Electric Company are the results of the continuing efforts of engineers, too numerous to list, who have studied and designed the Company's penstocks



JOINING SECTIONS



SEPARATING SECTIONS

FIGURE 9
JOINT ASSEMBLY

over about 35 years. It is their accumulated experience which has formed the subject of this paper. During the design and construction of what has been called the Company's modern penstocks, the Chief Engineer was Mr. I. Cleveland Steele, M. ASCE, who, upon retirement from the Company, was succeeded by Mr. Walter Dreyer, M. ASCE, the present Chief Engineer.

Paper 1398 October, 1957 PO 5

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

LARGE SPIRAL CASINGS OF T-1 STEEL[a]

E. L. Seeland,[1] M. ASCE
(Proc. Paper 1398)

ABSTRACT

Four of the largest turbine spiral casings field welded use the relatively new T-1 steel with reduction of cost due to less field welding and savings in erection time. These will be the first casings to date using T-1 steel, a low-carbon, quenched and tempered alloy plate steel with a high yield strength of 90,000 psi.

Four of the largest hydro-electric turbine spiral casings in the United States will be field welded using the relatively new T-1 steel.

The project is being constructed for the Washington Water Power Company and is known as the Noxon-Rapids Hydro-Electric Development (Fig. 1). It is located on the Clark Fork River about 3 miles southeast of Noxon, Montana, 20 miles upstream from the existing Cabinet Gorge Development of the Washington Water Power Company, and about 38 miles downstream from the Tompson Falls Development of the Montana Power Company.

At the Noxom-Rapids project four Francis Type Hydraulic Turbines (Fig. 2) will be connected directly to electric generators of 100,000 kw capacity each. The direction of rotation is clockwise looking downwards and the turbines are rated at 152 feet net effective head.

The units are numbered one to four with number five being the future unit. The normal pool elevation is 2,331 feet with an operating minimum tailwater at elevation 2,172 feet.

The inlet diameter of each spiral casing (Fig. 3) is 24 feet and the casings weigh approximately 108 tons each. The casing is designed to withstand maximum design pressure of 85 psi without exceeding unit stress of 27,500 pounds

Note: Discussion open until March 1, 1958. Paper 1398 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 5, October, 1957.

a. Presented at a meeting of the American Society of Civil Engineers in Buffalo, N. Y., June, 1957.

1. Superintendent of Design, Ebasco Services Incorporated, New York, New York.

psi on any net section with an assumed 90% weld efficiency.

These spiral casings were originally designed at ASTM 285 Grade B Mild Steel with maximum thicknesses of plates equalling 1-5/8 inches. With the use of T-1 steel the maximum thickness would be reduced to 1 inch.

Over the past few years, the author's company has changed from riveting to field welding of the casing steel plates, but the company had continued its search for a practical, high-strength, weldable steel to reduce the amount of field welding. Although no casings to date have been constructed using T-1 steel, it was found that this material has excellent properties of notched toughness at low temperatures with very high strength and good welding properties.

The T-1 steel is a low-carbon, quenched and tempered alloy plate steel with a high yield strength of 90,000 psi which is approximately 3 times the yield strength of ordinary carbon steel. The tensile strength of T-1 steel is 105,000 psi minimum.

The advantages of using T-1 steel are: less thicknesses of material to be welded in the field; savings in freight; savings in shop work and handling; and savings in erection time. The disadvantages are: additional cost of material; and special precautions necessary on attaching T-1 steel to the low-carbon steel stay ring and attaching T-1 steel to the low-carbon steel penstock.

It is very difficult to arrive at an accurate comparison of costs of constructing these penstocks of T-1 steel with construction costs using mild steel. However, the following information was developed to assure that a savings should result in the use of T-1 steel, taking all factors into consideration.

Using a factor of one for the relative cost of shop welding of ASTM 285 steel, the following relationship would apply on the same thicknesses of materials.

	ASTM 285	ASTM 212	T-1
Shop Welding	1	1-1/4	1-1/2
Field Welding	2	2-1/4	2-1/2

From an additional source it was estimated that the field welding of T-1 steel would cost 20% more than the welding of mild steel of the same thickness if both are fully X-rayed.

It is estimated that the erection and welding time per casing will be reduced approximately 25% using T-1 steel in preference to mild steel.

There is every reason to believe that the end product using T-1 steel would show a savings over mild steel. However, added precautions are being taken which could increase the cost because this T-1 steel has never been used in a spiral casing before. Even if the total cost between these two materials were equal, it would be advantageous to use T-1 steel because of its excellent properties of notched toughness and good welding properties.

All plates will be bent cold in the shop, and owing to the high yield strength of this material, extra heavy presses are required to form these plates. Each casing will be completely assembled before shipment and all pieces will be adequately match-marked. Special care is taken in the joint alignment to provide exact clearances for welding as few adjustments can be made in the field. The individual plates and plate sections of the knocked down casings will be adequately braced for shipment to destination.

The only heat treatment required in the shop is the tapered T-1 transition piece shop-welded to the heavy low-carbon steel stay ring (Fig. 4). These pieces will be furnaced stressed relieved after shop welding, at temperatures not over 1150°F. A tapered transition ring of T-1 steel approximately 12 inches long will be provided for field welding between the T-1 steel spiral casing and the low-carbon steel penstock (Fig. 5). Since the casing will be 100% X-rayed in the field and pressure tested, the shop will furnish an annular bulkhead for the stay ring and spiders in the lower stay ring, and one T-1 steel bulkhead to be used for test purposes on all of the four casings.

The spiral casing will be completely reassembled in the field before any welding is started. In assuring accurate alignment of the final section, the field welding sequence plays an important part. After the longitudinal joints in two adjacent courses are welded, the connecting girth joint can be welded. No girth joints are to be welded before the longitudinal welds. All other girth seams are to be welded complete before starting to weld the final girth seam near the nose piece. All welding joints on the scroll case will be done before welding the scroll case to the stay ring.

A turbine head cover will be placed in the stay ring and a spider placed in the discharge ring to minimize distortion of the stay ring during field welding and pouring of concrete.

In welding the spiral casing to the stay ring, a sequence of welding will be followed which will result in a minimum residual stress or distortion. The welders will be placed equidistant around the stay ring and will weld in the same direction simultaneously.

Welders are to be qualified according to section 9 ASME Code using T-1 steel in the qualification test. Only qualified welders will be used on any parts added to the casing that will remain part of the completed structure.

The weld joint detail to be used will be a single bevel with 45° included angle, with 1/8" shoulder and 1/8" gap for material thicknesses up to and including 9/16". Over 9/16" the detail to be used will be double bevel with 60° including angle with 1/8" shoulder and 5/32" gap. All bevels are made so that field welding can be performed in the down hand position using backup strips. Any backup strips that become part of the completed structure will be made of T-1 steel. The top half of the casing will be welded from the outside and the lower half of the casing will be welded from the inside. In most cases, the backup strips will be removed and the under side of the welds air arc gouged to clean base material before the final weld is applied. Air arc gouging will be used exclusively and no flame gouging will be permitted. Preheat of 200°-300°F will be used and this temperature will be maintained while welding is in progress. Hand pyrometers or tempil stiks will be used to measure the temperature. The welds will be rough ground and all butt-welded joints of the casing shall be examined by X-ray. Weld reinforcement will be limited to 10% of the thickness of material being welded.

Low-hydrogen electrodes have coatings that absorb water from the air, and the absorption of a small amount of water will have a harmful effect on the operation of the electrode and quality of welds made by them. All electrodes will be removed from containers and baked at 850°F for one hour before using, then stored at 250°-350°F. Only the amount of electrodes that can be used in two hours will be removed from the storage heaters. All unused electrodes will be re-baked at 850°F for one hour and the same electrode should not be baked more than four times.

The weld rod used will conform to the AWS specifications for E10016 which

has physical properties similar to T-1 steel. The welding rod and the diameters used should be as established in the procedure qualification. The sizes of rods will be used in the positions in which each diameter is applicable, namely 1/8", 5/32", 3/16" and 7/32" diameter rods.

After the spiral casing has been completely erected and welded, it will be subjected to a field pressure test before any concrete is placed around it. The test pressure will be 50% in excess of the design pressure.

Inspection of the welding will be made at all times welding is being performed, by qualified, experienced inspectors under the control of the purchaser.

The design and construction of the installation is by Ebasco Services Incorporated.

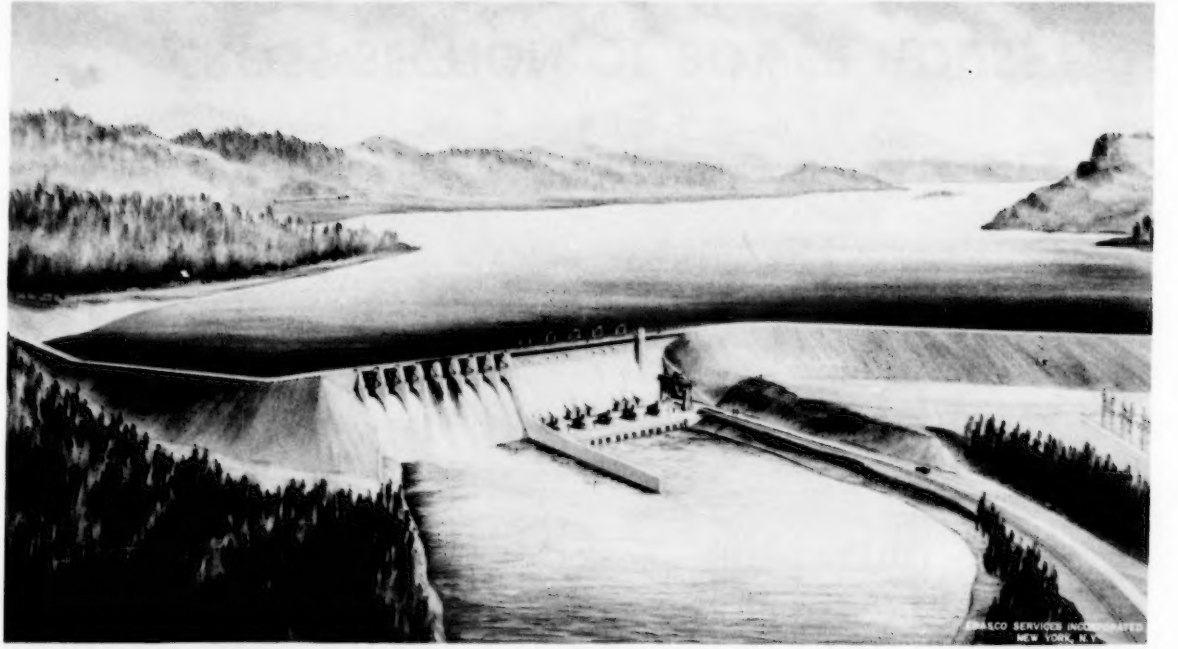


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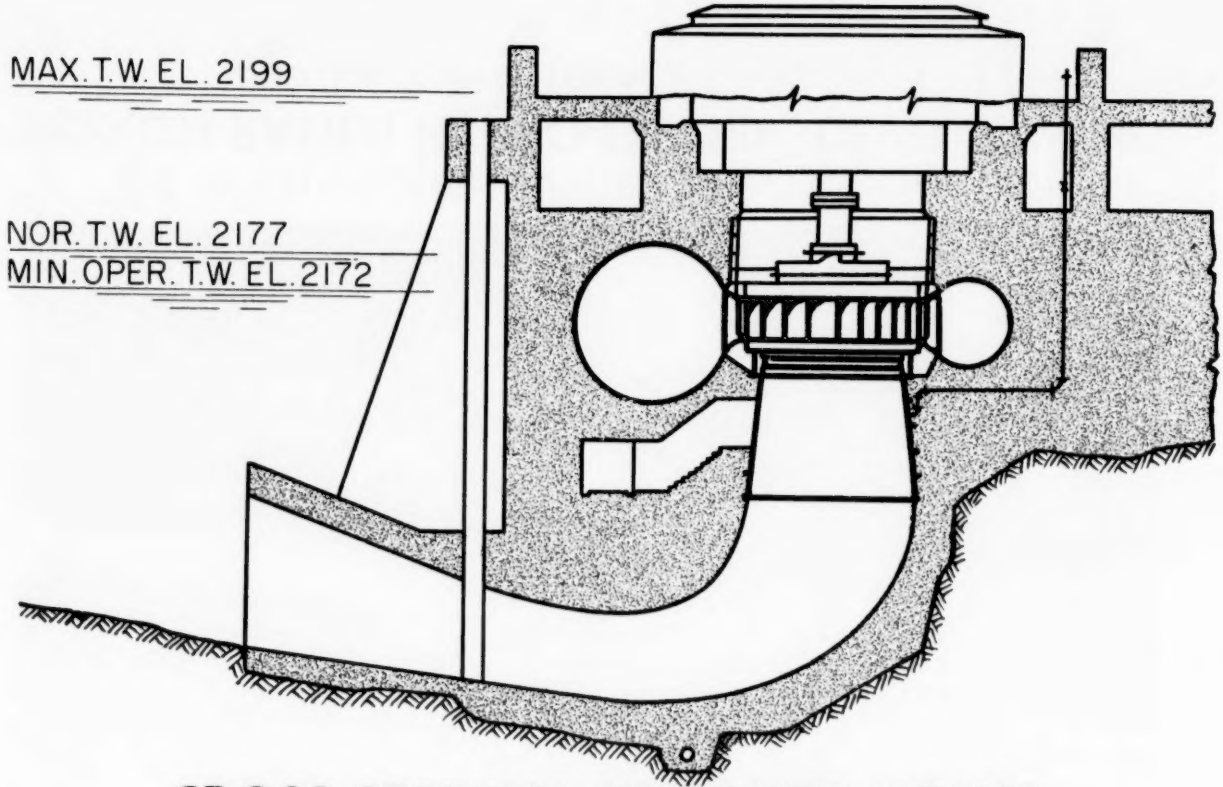
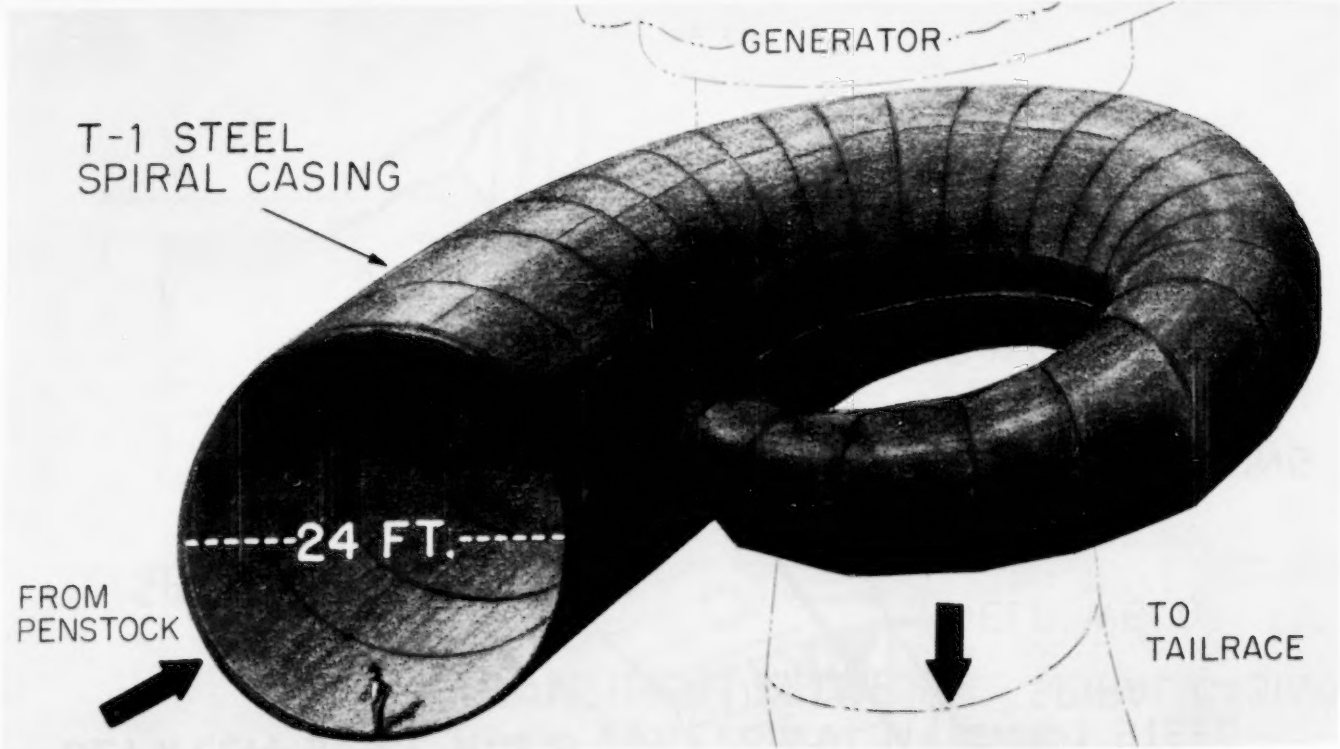
Fig. 1.



CROSS-SECTION OF POWER HOUSE

Fig. 2.



TYPICAL T-1 STEEL SPIRAL CASING

Fig. 3.

TRANSITION BETWEEN STAY RING AND SPIRAL CASING

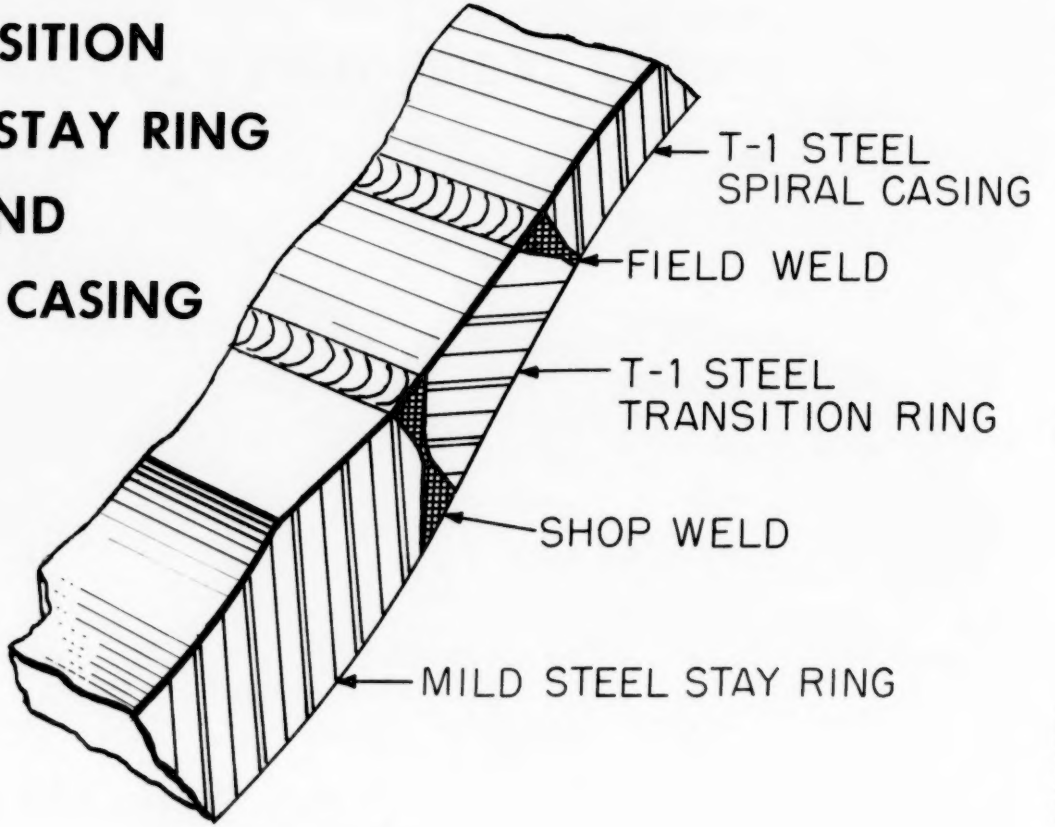
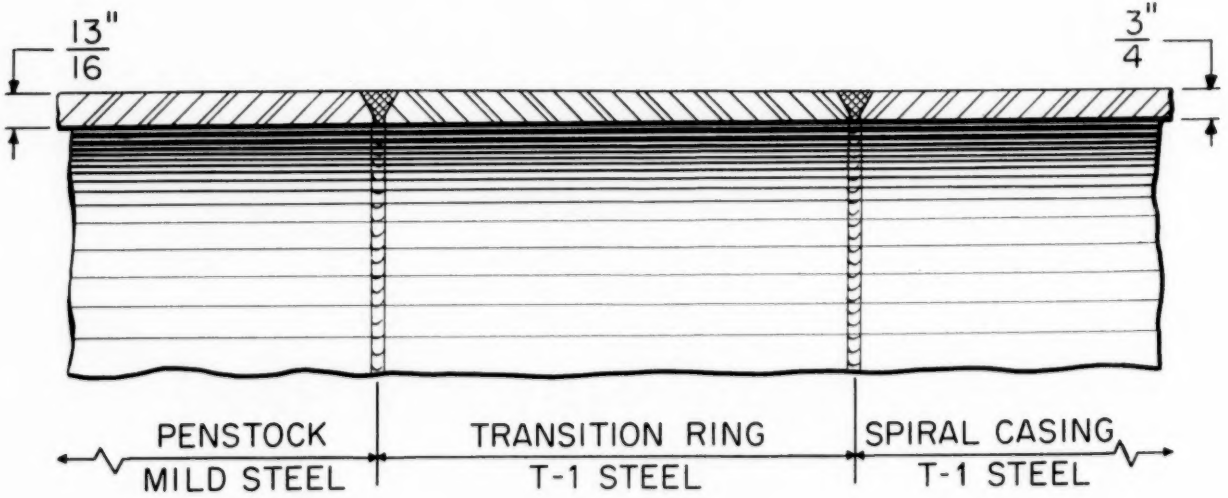


Fig. 4.



TRANSITION BETWEEN PENSTOCK AND SPIRAL CASING

Fig. 5.

Paper 1414 October, 1957 PO 5

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Proceedings of the American Society of Civil Engineers

COEXISTENCE OF FISH AND DAMS[1]

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(Proc. Paper 1414)

SYNOPSIS

The strong interest in water resource development throughout the nation imposes a moral obligation as well as an economic necessity to provide for the preservation and improvement of the great natural fishery resource, which exists particularly in some of our Western rivers. This paper outlines the background of the great Columbia River salmonoid fishery, the problems of sustaining it in the face of intensifying watershed development, research that is being done to maintain adequate migratory routes and spawning conditions, and the need of continued progress toward improvement of the fishery in light of the program for river development that is being undertaken.

INTRODUCTION

The authors have undertaken preparation of a paper on the subject of fish and dams only with some hesitancy in view of the controversial aspects commonly attributed to these two subjects as associated with each other. Being engineers and not biologists, they have touched on the biological aspects

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1. Presented at a meeting of the American Society of Civil Engineers in Pittsburgh, Pa., October, 15, 1956.
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of fishery only to the extent necessary to present a general background of the problem. In these phases the authors make no claim of authoritativeness, and they welcome constructive discussion, and criticism if need be, from those who are authorities in this field. Opinions expressed and conclusions drawn are those of the authors, and do not necessarily represent official views of the Corps of Engineers.

The primary objectives of this paper are to present constructively, from the engineers' point of view based on long association, the origin, nature, and possible solutions of one of the most difficult and complex problems encountered in the development of Pacific Northwest rivers, commonly referred to as "fish versus dams." The authors will endeavor to show, however, that this picturesque title is now outlived and should be changed to "fish and dams," thanks to a cooperative spirit which is evidenced among nearly all who are concerned with achieving a positive solution. After a description of migratory fish runs in the Columbia River and the general impact of the works of man on the fishery resource of the Columbia, the paper discusses the early efforts towards amelioration of the adverse effects, and the measures undertaken by the Corps of Engineers and others to modify project plans and to provide adequate passage and/or replacement facilities at major dams on Columbia River and its tributaries. It outlines the research program sponsored by the Corps and generally executed by the fisheries agencies themselves, directed toward analysis of the basic problems and to evolve scientifically sound approaches for the design and construction of effective fish passage facilities; together with parallel investigations and research by the fisheries agencies and by other construction agencies. Some of the accomplishments up to the present time are cited. The paper records real progress toward the long range objectives of "live and let live" implied in the title, being achieved by a cooperative approach to the conservation and development of <u>all</u> water resources including migratory fish.

Columbia River and Migratory Fish

The Columbia, one of the great "resource" rivers of the United States, stands at the head of the list in hydroelectric power potential, and its migratory fish resource is exceeded by few other streams in the world. (See Chart 3) The maintenance and possible enhancement of this fishery in the face of rapidly expanding harnessing of the river for flood control, irrigation, navigation and power is a problem of vital importance to the region. It is, moreover, a problem which in the past has generated heated controversy between opposing interests. The coexistence of fish and dams has been, and is, a major challenge to biologists and engineers. "Dams" has often been a fighting word to the commercial and sports fisheries, and, understandably, has often raised unyielding opposition to plans for river development appearing to threaten their interests. On the other hand, to many segments of Northwest industry, commerce, agriculture and to many cities and communities, "dams" signify progress, new resource development, industries based on low cost power and river transportation, irrigation of arid land, protection from floods and generally a higher standard of living. Nowhere else in the United States has this conflict of interests become so acute. In recent years, however, the attitude of irreconcilable conflict between these two forces has tended gradually to yield to a spirit of mutual understanding and to an approach of mutual cooperation.

Civilization appears often as the inherent enemy of wildlife, and advancing civilization has taken a heavy toll on the salmon streams of the Northwest. However, in recent years man has recognized the peril and has taken resolute steps to remedy early errors. These measures are being undertaken on a cooperative basis by many agencies—Federal, State, and municipal, as well as public and private utilities—and involve great costs. This paper, after outlining the background of the problem, will describe in brief form some of the many facets of this cooperative program.

White man, on his arrival to the Pacific Coast of North America more than one hundred and fifty years ago, found the coastal rivers teeming with salmon —a delectable fish which provided a staple food supply for the native Indians. The Lewis and Clark expedition of 1805-06 to the Pacific Northwest found Indians by the thousands migrating annually to the shores of the Columbia to gather their winter supply of salmon which they preserved by drying in the sun. Dried salmon was a prime item of trade between river tribes and those of the interior and it is estimated that the annual harvest totaled 18 million pounds. Early settlers soon discovered the value of the salmon, and in 1866 a commercial cannery was established at Astoria on the mouth of Columbia River. By 1883 the commercial salmon fishery landed almost 43 million pounds. That is probably the peak production but very high productivity was maintained until the 1920's when the detrimental effects of encroaching civilization in combination with the exploitation of this great resource became a problem of concern. The Columbia River's annual salmon pack in 48-pound cases since 1888 is shown in Chart 1, and for comparison the pack from Alaska waters. There is, in addition, today a world famous sport fishery on the Columbia and other streams of the Pacific Northwest.

The anadromous or migratory fish of the Columbia River consist of steelhead trout and four species of Pacific salmon. The steelhead trout, somewhat similar in characteristics to the Atlantic salmon, is both a highly prized sports fish as well as contributing to some extent to the commercial fishery. Living much of their adult life in the ocean, they migrate into the rivers to spawn. The adult steelhead entering fresh water from the ocean average about 10 pounds in weight and may reach 40 pounds. These fish, in contrast to the true salmon, do not necessarily die after spawning. Due to their vigor and migration habits, this specie has been quite successful in withstanding the encroachments of civilization.

The second and more important group of anadromous fish entering the Columbia River is the Pacific salmon, of which there are four species: the large chinook or king, averaging 20 to 30 pounds and attaining up to 100 pounds in weight; the blueback or sockeye averaging 3 to 4 pounds per fish; the silvers averaging 10 pounds; and the chums weighing about 12 pounds. Records indicate that the Pacific salmon originally migrated into practically every tributary of the Columbia River system from the sea to the Rocky Mountains 1,000 miles inland, and from British Columbia to Southern Idaho. They are also found in great numbers in the coastal streams of Oregon, Washington, British Columbia and Alaska. The salmon, after spending their adult life in the sea, 2 to 5 years depending on their characteristics, invariably return to the parent stream to spawn, and there to die. On entering fresh water they proceed, at the rate of about 10 miles per day, upstream to the specific tributary and particular spawning area in which they were hatched. Here in the gravels of a shallow stream, the female scoops an oval cavity and in this next deposits her eggs to be fertilized by the male. Exhausted, the

parent fish die after spawning. Each female lays 2,000 to 5,000 eggs. The time required for these eggs to hatch varies from 50 days up, depending on the water temperature. The newly hatched fry begin life in the gravels where they were born, and later, as "fingerlings", move into the open stream to live in fresh water for varying periods, with the majority migrating to the sea in their first year, or the spring of their second year.

Each tributary stream supports a separate population or race which migrates inland from the sea at a time normally most favorable for reproduction in that stream. These separate races have developed individual inherited characteristics favorable for their survival. The Pacific salmon normally do not feed after entering fresh water to spawn, hence, those fish utilizing spawning areas located long distances from the sea must have the ability to store sufficient energy for their travel and furthermore must start their spawning migration at the correct time to reach these spawning areas for reproduction during optimum conditions for egg and fry survival. Any unnatural delays en route may therefore become critical in maintenance of the fish runs. The strong instinct to return to their parent stream is another important factor in maintaining the fishery, since a blocked migration route usually means loss of that race because they will normally die without spawning in attempting to find their usual habitat. A third major factor in the survival of this fishery is the maintenance of the spawning areas in their natural and undisturbed condition. Favorable spawning gravels must be of the proper size, gradation and depth, free from silt or other deposits, and with hydraulic conditions creating a movement or gentle flow of water through the gravels; the water must be of proper temperature and free of injurious polutions. A major change in the regimen of a stream means a major change in the segment of the salmon run using that stream. A fourth significant factor in the maintenance of the salmon fishery is the condition of the downstream migration route to the sea. The principal migration of the young fish occurs with the spring freshet or higher river flows, the fingerlings typically drifting downstream, tail first. Major obstacles often cause mortality, or stunning which makes the fish easy prey for predators.

Effect of Works of Man

Progeny of the salmon which spawn in the Columbia River and tributaries are subject to commercial troll fishery in the ocean along the Pacific Coast from southeastern Alaska to northern California. They are further subjected to an intense fishery, both sports and commercial, through the first 200 miles of Columbia River when making their spawning migrations. The value of the Columbia River segment of the Pacific Coast salmon fishery is estimated at close to $17,000,000 annually, based on first wholesale cost of the commercial catch plus the estimated expenditures for the sports catch, arrived at by surveys of the fishery agencies.

Coincident with this exploitation of the fish population is the economic development which has taken place during the past eighty years in the Columbia watershed, creating conditions adversely affecting the productivity and seriously threatening the survival of anadromous fish. Early mining blocked or diverted smaller streams, and mining wastes destroyed spawning beds and young fish. The lumber industry cut over large areas of forests having fundamental effects on the streams draining those areas. The elimination of trees

and shrubs changed the temperature of water and the flow regimen; logging operations in some instances blocked the streams or filled them with debris. Similarly, agricultural development changed the regimen of stream flow with heavy summer demands for irrigation water requiring dams and diversion works. All of these conditions completely changed the runoff pattern for unnumbered tributary streams which are the principal spawning areas for the salmon. Denudation of lands were conducive to floods which caused erosion, siltation, and too high flows for fish passage, while dams and diversions left the streams with too little water during summer and fall months. Unscreened irrigation intakes took their toll of fingerlings. Other aspects of advancing civilization were equally destructive. Wastes from industries, canneries, pulp and paper mills, and from towns and cities along the river banks, polluted some streams to the point that fish life could not exist during the low water season.

Later large scale development of irrigation and hydroelectric power on the main stems and major tributaries required increasingly larger and higher dams, impassable for upstream migration without fish passage facilities, and some with almost impossible conditions for downstream migration of fingerlings either through the power plants or over the spillways. Grand Coulee Dam on the Columbia River is probably the most spectacular of such structures where the entire river system above the dam has been closed to salmon migration, although equivalent runs have been sustained below the dam by means of extensive rehabilitory works including fish hatcheries.

These changing stream conditions and the structures of man, however, can not be blamed entirely for the decline of the anadromous fishery. Convincing evidence in this regard can be found on streams subject to sports and commercial fishing outside the Columbia Basin, where natural conditions still exist undisturbed and yet the salmon runs have declined proportionately as much as they have on the Columbia. This is especially true on certain undeveloped coastal streams in Oregon and Washington, and to a substantial extent true in Alaskan waters whose potential salmon yields far exceed those of the Columbia River. The spawning rivers of Alaska in particular are virtually untouched by man except for commercial fishing in adjacent coastal waters and yet the salmon catches during the past 20 years have declined drastically from 8,455,000 cases in 1936 to 2,470,000 cases in 1955. Reference is made in this connection to Chart 1. Whether this drastic decline is due to overfishing or to other possible causes has not been established; neither can it be determined with certainty to what extent similar causes have affected the Columbia. Many believe that over-fishing, ocean catch of immature salmon, and depletion of salmon food have been factors of great importance.

Remedial Measures

The serious effects of encroaching civilization on the salmon fishery began to be realized even prior to the beginning of this century and during succeeding decades various remedial measures were initiated through specific works and legislative acts by the states. The measures, generally concerned with regulation of fishing, are summarized briefly as follows:

a. Regulate fishing gear.—The picturesque giant fish wheels were prohibited; size of mesh in gill nets were regulated to permit escape of particular species; and use of the sometimes mile-long seine nets on the lower Columbia banned.

b. Regulate fishing season.—Commercial fishing seasons have been progressively restricted until at the present commercial fishing on Columbia River is prohibited during about 20 percent of the possible 5 to 6-month period.

c. Regulate fishing areas.—A number of Oregon streams have been closed to commercial fishing, and Columbia River will be closed to such fishing above Bonneville Dam (145 miles from the mouth of the river) in 1957.

d. Restrict ocean catch of immature fish.

e. Screen irrigation intakes.—State legislature now prescribes screening of any new stream diversion and state agencies are screening old diversions within the limits of available annual funds.

f. Install fish ladders.—State laws prescribe adequate fish passage facilities at all structures.

g. Remove river obstacles.—State agencies are empowered to do clearing and snagging in stream channels and provide fish ladders at natural obstructions.

h. Pollution abatement.—Pacific Coast states, Oregon and Washington in particular, have adopted increasingly strict legislation against the dumping of untreated municipal sewage and industrial wastes into flowing streams.

Structures.—Prior to the 1930's very little structural work had been done to preserve the anadromous fishery in the Columbia Basin. Small fish ladders had been built into several low-head power dams, some of which were on such large rivers as the Clearwater and the Willamette; unfortunately, many of these ladders proved to be relatively ineffective because of inadequate knowledge at that time of design requirements. Some other modest attempts were made to correct stream conditions particularly for irrigation, such as placing small ladders over diversion dams and screening canal intakes. Other than these preventive measures, principal efforts were directed toward control of the fishery by means of regulatory measures on the catch, but doing little in the way of research and planning for the future.

In the 1930's, however, several major fishway installations were initiated. Completion in 1933 of the first dam across the upper main stem of the Columbia, at Rock Island, 500 miles above the mouth, also marked the initial operation of the largest fishway installation up until that time; and the first installation on so large a river. These ladders, 20 feet wide with a slope of 1 on 10, and designed to pass the predominant blueback runs over the 54-foot high dam, have now, with considerable modifications, functioned fairly successfully over nearly a quarter of a century. The second major fishway installation, on much larger scale than at Rock Island, was made by the Corps of Engineers in the construction in 1934-1938 of Bonneville Dam, located at the head of tidewater 145 miles inland from the mouth of the river. Here competent biologists were employed as well as engineers and attempts made to analyze the action of adult fish, collect them at strategic points and pass them upstream over the 62-foot high dam with a minimum of delay and through adequate passageways. Very little scientific knowledge was at hand on fish behavior and needs, so empirical criteria had to be developed. Ladder widths of 30 and 40 feet were adopted, with slopes of 1 on 16. The fact that this planning was good is attested by the passage records these 18 years averaging 650,000 salmonids annually, and the absence of any major evidence of loss, blocking or delays at the dam. (See Chart 2)

An event of major importance following the construction of Grand Coulee

Dam, concurrent with Bonneville, was the transfer and rebuilding of blocked blueback runs into tributaries downstream from Coulee. To complicate the problem, in 1941 disease practically destroyed the blueback run of that year to the area. By a major program of protection of adults and propagation by replanting of fingerlings in the lakes of the Wenatchee and Okanogan River basins, the runs have been revived and restored to a large extent. This can rightfully be considered a major triumph for the principle of enlightened fishery management.

After Bonneville and Grand Coulee Dams were under way and until the early 1940's, it was the opinion of many that it would be a long time before any further developments of such magnitude would be justified on the Columbia. Consequently, during this interval very little more was done to consider the basic problems or future prospects for the salmon. However, by 1942 it was evident that early development of additional major power sources was essential to meet the growing Northwest power needs, including the booming aluminum industry. No starts were possible during World War II, although major projects were authorized by Congress including McNary and Chief Joseph dams on the Columbia and four large navigation-power dams on lower Snake River.

Full scale planning for McNary Dam started at the war's end, with construction initiated in 1947. Again engineers and biologists were faced with the problem of passing over half of the entire Columbia River fish run over the dam, both during the difficult construction phases and over the permanent structures. This dam had a maximum effective height of 92 feet, 50 percent in excess of Bonneville. After taking inventory of known facts and conditions it was found that little additional knowledge had been developed since the construction of Bonneville Dam except for the experience gained in operation of the project and the research done for the Hells Gate problem on the Fraser River (see below). McNary Dam was not only higher than Bonneville but considerably larger, with problems of passage proportionately increased. For some of the design problems, special studies were made of the Bonneville facilities, and numerous model studies made at the Bonneville Hydraulic Laboratory, checking flow conditions, weir design, structural design, and many other features. In developing plans for McNary, certain improvements were made over the Bonneville facilities and some new features added, such as automatic control of water supply, pumped water for attraction flows, and automatic operation of fishway entrance facilities. Final McNary plans included two fish ladders 30 feet wide with 1 on 20 slope; a fish collection system at the powerhouse with an additional interior transport channel; an atmospheric pressure fish lock; and a high pressure fish lock. The fish locks were installed for experimental purposes, and as alternative means of passage in lieu of the long, expensive fish ladders. The fish passage facilities at McNary Dam are the subject of a paper prepared for the Journal of the Power Division of the American Society of Civil Engineers and is published as Proceedings Paper Number 895, Volume 82, PO1, dated February 1956. This paper contains the basic criteria for fishway design and a description of the many features of the facilities.

About this time there was recorded on Fraser River in British Columbia another milestone of greatest significance to the salmon fishery. Prior to 1913, the Fraser salmon fishery was second only to Alaska in importance to the industry. In that year, in connection with railroad construction operations,

there occurred at Hells Gate a slide which partically blocked the river to salmon runs. Subsequent efforts were made to remove the slide with assumed success. However, the Fraser River fishery over the succeeding decades declined to a mere fraction of its former magnitude. In the 1940's this problem was subjected to intensive studies and experimentation by the International Pacific Salmon Fisheries Commission. It was found that a block still existed at certain river stages which are frequent during major runs. As a result, a new type of fish ladder was developed and completed in 1946 at Hells Gate. This Hells Gate ladder depended on the principle of vertical slots rather than on weirs with shallow overflow, and consisted of baffles placed vertically on alternate sides of ladder walls. This type of ladder will function effectively through a great range in depth when tailwater and headwater fluctuate approximately the same amount. This ladder was immediately successful in relieving the fish block on the Fraser, and by 1954 the fish run had increased dramatically to the largest volume since 1913, giving evidence that complete restoration of this once great salmon stream was a possibility.

By the time that McNary Dam was on the drafting boards the Corps was directed to make a complete review of its earlier comprehensive report on development of Columbia River, the so-called Columbia River "308" Report of 1931. This review, completed in October 1948, presented a greatly expanded program for development of the water resources of the Columbia Basin. Its "main control plan", a program of development anticipated to be completed by 1975, envisioned two more Columbia River dams similar to McNary to be built at The Dalles and at the mouth of John Day River. The plan also included a major dam at Priest Rapids above the mouth of the Yakima and the Hells Canyon project of the Bureau of Reclamation on the middle portion of Snake River. (See Chart 3) Most of these projects, which were only a part of the main control plan, are astride the principal paths and spawning areas of the salmon. In addition, and situated on the main migration route of the Snake River chinook runs, were the above mentioned four navigation-power dams on lower Snake River between its mouth and Lewiston, each of them similar in height and features to the McNary project. The Columbia and lower Snake River projects, when completed, would comprise an almost continuous system of dams and slack-water pools extending 205 miles on Columbia River from Bonneville and 140 miles on Snake River. (So rapid has been the progress of dam building that by 1957, only 9 years later, four of nine dams—or alternates thereto—are under construction, with The Dalles and Chief Joseph nearing completion. See a later section of this paper.)

Fishery Development Program.—In preparation of the comprehensive report of 1948 on the Columbia River basin the Corps enlisted the cooperation of many interested agencies. Included was the Fish and Wildlife Service which joined in studies of water resource developments as affecting fish and wildlife of the basin. As one of its major findings the Service recommended that major steps be taken, primarily in the lower river and its tributaries below McNary Dam, partially to compensate for some of the losses that were found to occur in the upper river areas. This Lower Columbia Fishery Development Program, adopted by the Corps as a part of its comprehensive report, recommended improvements along certain tributaries which were known to be valuable to the salmon industry, to develop their optimum capacities for salmon reproduction. Log jams were to be removed, difficult falls or rapids laddered and diversions screened. In addition, a major hatchery

program was recommended for the artificial propagation of salmon. This entire program was accepted by Congress in 1950 and the work has been under way during the past six years. Approximately 15 million dollars have been spent and good progress realized on many of these projects. Eventual expenditures up to about $40,000,000 are programmed.

The findings of the Columbia River Review Report made it evident not only to the Federal Government but to the states involved and to private industry that a major program of research and constructive rehabilitation must be undertaken immediately to keep up with the progress in other phases of water resource development in the Columbia Basin. Research work has been going on, especially in basic ecology by the U. S. Fish and Wildlife Service, the University of Washington and to a limited extent by the state agencies. However, a much expanded program of study of fish behavior, their migrating and spawning needs, their action at barriers, and methods of guiding was needed. Some of these basic problems were under study by the International Pacific Salmon Fisheries Commission as a continuation of the Fraser River development in Canada. Federal and State agencies gradually increased the tempo of their studies and in 1953 a cooperative agreement was reached between the Corps of Engineers, the three State Fish and Game agencies involved, the U. S. Fish and Wildlife Service, and the University of Washington, for a much intensified research program. Much of this program was financed by appropriations to the Corps of Engineers administered through the medium of contracts with the several agencies for specific studies. Each of the agencies within its own limitation of funds stepped up its program study to complement the Corps program. This tempo has increased until at the present time there are probably thirty to fifty individual studies being conducted in relation to river development and the salmon of the Pacific Northwest.

It is interesting to note the change of emphasis in this research program. In the planning of McNary Dam the emphasis was placed primarily on requirements for adequate passage upstream of the adult salmon migrating to their spawning beds. The engineering and biological studies of the problems of attracting and collecting of upstream migrants and affording them safe passage up over the dams of almost any height, by ladders for the lower dams and by locks or tanks on inclined tramways for higher dams, have been effected to a degree where it can safely be said that a high structure need no longer be a major obstruction to the upstream migration of adult fish. Problems do still exist for optimum upstream passage, and for reduction in the present extremely high costs for these facilities. Among such factors may be mentioned widths and slopes of ladders, amount of water supply, and further development of locks for large scale movement of fish. Nevertheless, the major problems now lie in developments for safe passage downstream of the fingerlings migrating to the sea. This is evidenced by a review of the several studies being made by the fishery agencies. Fourteen separate studies have been made in the past five years on the problems of downstream migrants at dams. Another fifteen studies have been made by the agencies on the guiding of downstream migrants. Other studies relate to the biological aspects of the salmon, water problems of flow and quality, problems of adult salmon at dams, fishway studies and migration of adult salmon.

The many studies concerning guidance of downstream migrants presents an interesting review of this facet of the research program. Guidance by artificial means of all types is being investigated. Electrical guidance in

many forms and under many conditions is receiving extensive studies. Parallel to those studies must be the other investigations of the effect of electricity on fingerlings; the effect of electric shock upon reproductivity and the limiting ability of various sizes of fish to withstand electrical currents. Similar studies are being made on light as a means of guidance and the effect of light waves on migration. Sound waves and chemical changes of water also affect fish and these factors are being studied. With all of these artificial directional studies must go physiological and phychological studies regarding their effects.

In investigating the problems of downstream migrants, the distribution of fingerlings in natural streams and in reservoirs must be determined, including not only the depth at which they travel but the depth to which they will sound when a partial barrier is encountered. When fingerlings reach dams or major obstructions the various avenues of migration past that structure must be investigated, including the relative desirability of passing over the spillway; through the power turbines; or by specially designed bypass routes. Various types of spillway design are studied including mortalities under a range of operating conditions. The hydraulically desirable backroll of a spillway to break up high velocity can be very damaging to downstream migrants. On higher dams the path of the water down the face of the spillway is an important feature and recent studies on modified spillways show possibilities for a ski-jump type where it has been shown that fingerling fish experiencing a free fall with terminal velocities, plunging into a pool of water below the dam can survive with very low mortality rates. Evaluation of mortalities caused by squaw fish and other predators on young salmon is a subject of parallel study. Marked fingerling by the hundred thousands are being placed under all kinds of prototype conditions at many types of structures to study their relative survival rates, and effect of passage conditions on them. Effect of pressure changes as they pass through turbines and sometimes enter areas of negative pressure are being investigated.

Condition of the water in the stream is an important feature of research for the fishery. Water quality, of course, has a direct relation with respect to pollution. The temperature regimen of the river also has a very profound effect on migrating salmon. Low flows with high temperatures encourage disease, and reduce stamina. Waters taken from the bottom of a reservoir are colder and again change the action of salmon, even increasing spawning below the dam because of this temperature regimen. Water flows and action of water through spawning gravels is an important feature and the success of artificial spawning areas may depend on proper percolation. One of the important aspects of this research program is the fact that study of a particular condition at a dam or in a stream raises specific biological, physiological and stream flow questions which must be answered by extensive laboratory investigations before proper evaluation of the particular problem can be made.

The extensive research program indicates the determination of those planning for the development of the water resources of the Columbia Basin to assure the continued existence of this great salmon resource along with the flood control, navigation, irrigation and power development necessary for the economic progress of the region. To illustrate the extent of this research and development program for the fishery, and the widespread participation in these cooperative efforts, there is cited below a list of the agencies actively contributing to these studies. The public agencies listed are largely the same

agencies through which the research program of the Corps of Engineers is being prosecuted.

Canadian Department of Fisheries
United States Fish and Wildlife Service
Fisheries Research Board of Canada
International Pacific Salmon Fisheries Commission
University of British Columbia
University of Washington
Washington State College
Washington State Department of Fisheries
Washington State Department of Game
Oregon Fish Commission
Oregon Game Commission
Idaho Department of Fish and Game
The City of Tacoma
Idaho Power Company
Pacific Northwest Power Company
Chelan County PUD
Grant County PUD
Pacific Power and Light Company
Portland General Electric Company

In addition to these agencies the industry has a research program of its own concerned primarily with the Alaska phases of the fishery. The several power companies and public utility districts are currently interested in power projects and are participating cooperatively with the State fishery agencies in several studies.

Tangible Achievements

With the fish and dam problem under attack on so many fronts and by so many agencies, what of the achievements to date? Has any real progress been made? The authors believe the answer should be in the affirmative, that findings and developments of greatest importance to the ultimate solution of the problem have been achieved. The more significant of these will be discussed briefly. Some of these forward steps are not sufficiently proven by experience to be accepted by all at face value, so may be subject to divided opinions; nevertheless it is felt that they should be pointed out as possibilities at least, subject to the tests of further experience.

Upstream Passage of Adult Salmon over High Dams.—Bonneville Dam fish ladders, with an average lift of 62 feet, have now been operating 16 years. Chart 2 of this paper indicates that, over the long term, escapement over the dam has been increasing. Whether this is due to increased production in the Columbia and its tributaries above Bonneville, or merely due to increasing restrictions placed on commercial fishing below the dam is a debatable question. Nevertheless this record does seem to indicate a considerable degree of success for this operation. McNary Dam fish facilities, with their 50 percent greater height and $28,000,000 cost, have been in operation only since 1953, a period too short to draw any firm conclusions as to the effectiveness of these ladders. However, there are no indications of blocking or undue delay to the fish migrations due to this structure, and available evidence with regard to upstream passage, insofar as known to the authors, is all favorable.

Temporary means of passage by use of a V-trap and bucket was experienced at McNary Dam during construction. Fish were attracted into a holding chamber and thence into a 1500 gallon bottom dump bucket which was hoisted over the dam to the forebay by crane. The operations, because of temporary construction conditions were more complicated than would normally prevail; however, the operation was relatively successful, with as many as 15,000 salmon being passed in one day. From this experience it would seem entirely practicable to apply the bucket or elevator principle to a dam of any height. This basic principle of collection of the fish in a container which is then hauled up an inclined tramway, is used at the 230 foot high Baker Dam.

Downstream Passage of Fingerlings over High Dams.—Within the past few years a series of significant experiments related to the downstream passage of fingerlings over high dams have been conducted in the Northwest, principally by the Washington State Department of Fisheries. These are summarized as follows:

a. Spillway and Turbine Experiments—Elwha and Glines Canyon Dams.—At the 100-foot high Elwha Dam, fingerlings released over the spillway, which discharges on to rocks below the dam, showed only 62.9 percent survival. In another series of tests some 17,900 yearling silver salmon were released over the free overfall spillway of the 200-foot high Glines Canyon arch dam, with the favorable survival rate of approximately 92.2 percent. Release of 17,925 yearling silver salmon through a Francis type turbine utilizing 1500 c.f.s. under 184-foot head resulted in a survival rate of 69.6 percent. At the nearby Elwha Dam 37,000 yearling silver salmon were released through the Francis turbines, each utilizing 500 c.f.s. under a 100-foot head, with a survival rate of 100 percent. These tests indicate that, contrary to general impressions, fingerlings do have a fair chance of passing safely over high spillways and through relatively high head Francis turbines.

b. Free Fall Experiments in Air.—Experiments in free fall have recently been conducted with fingerling fish up to six inches in length. These fish were dropped from a helicopter at a height of 300 feet into a body of water with no mortalities. Wind tunnel studies showed that fingerlings may reach their terminal velocity in air after a fall of 150 feet. This principle of free fall has direct application to passage over spillways. It is put to practical use as an accepted procedure in planting mountain lakes by dumping of fish from an airplane.

c. Release over Ski-Jump Spillway.—Significant to the successful passage of fingerlings over dams have been recent experiments at the 230-foot high Baker arch dam (on Baker River, Washington) where 240,000 fingerlings were released over an improvised "ski-jump" spillway to drop into the pool below. A survival of 85 percent was obtained as compared to the survival rate of only 40 percent for those fish passing over the regular spillway and down the face of the dam. The relative success of the ski-jump experiment, together with earlier tests, indicates the possibility of passing fingerlings over any height of dam by means of a free drop provided the tailwater pool is of sufficient depth.

Downstream Migration.—Experiments have indicated that the greater number of the fingerlings moving downstream on a large body of relatively still water, such as a deep reservoir behind a dam, travel generally in the top 15 to 20 feet of water. This indicates that they are less likely to enter

the turbine intakes if the latter are located at a considerable depth below the surface. This combination of factors lends great significance to the current research program on fish guidance. The City of Tacoma, in connection with design of two high dams planned for Cowlitz River, a tributary of the lower Columbia, have developed experimentally a practicable means of separating fingerlings from the bulk water of the forebay. In collaboration with the Washington State Department of Fisheries, the City has devised a system briefly described as follows: Based on the hypothesis that fingerlings will be attracted to an outlet by a large flow near the surface, a "skimming" device or weir is located at the face of the dam, with attraction flow created by low head pumps. This water passes over screens which allow the fish and a small amount of water to pass over the top, from whence they are transported by an open, free flowing flume to the tailrace below the dam. If the parallel need for a practicable means of guiding the fingerlings to the weir can be developed by the current research program, there seems to be a distinct possibility of reaching an acceptable solution to the overall problem of downstream fingerling passage.

Artificial Propagation.—In recent years the possibilities of artificial propagation have been the subject of extensive experimentations, and a number of stations have been constructed for artificial spawning. For many years the feasibility of artificial propagation of salmon was in serious doubt and there are still many unanswered questions. However, the Fish and Wildlife Service and the States of Oregon and Washington, after persistent experimentation and progressive improvement of techniques, have demonstrated that artificial propagation for certain species is effective in building up existing runs, and also in establishing new runs where none existed previously. An exceptional example is the Spring Creek Hatchery on a tiny tributary of Columbia River above Bonneville Dam. On this stream, only 300 yards in length and fed by springs, the U. S. Fish and Wildlife Service has established a large hatchery for artificial spawning of chinook salmon and release of fingerlings. Here, where no natural salmon run existed, has now been established an annual return of 10,000 to 18,000 fish—a striking example of the instinct of the salmon to return to the very point of its birth. There are now about 20 salmon hatcheries operating in the Columbia River basin.

Replacement of Willamette River Natural Spawning Areas by Hatchery Operations.—Artificial propagation by hatcheries has recently been applied in the Willamette Basin and elsewhere with the intent to maintain and build up salmon runs cut off by high dams. Detroit Dam, about 368 feet in effective height, and Lookout Point Dam, 230 feet effective height, have been constructed across the North Santiam and Middle Fork Willamette Rivers respectively. In both instances these heights were considered at the time of construction to preclude passing of fish over the dams. Instead, the Corps of Engineers from project funds financed the construction of large hatcheries on each stream to furnish replacement stock for the runs cut off from upstream spawning areas, to be operated by the Oregon State Fish Commission. Adult fish are taken in the main streams downstream from the dams and transported to the hatcheries for artificial spawning. The capacity of each hatchery is 14,000,000 eggs annually.

Artificial Spawning Grounds.—A new technique under active consideration for application to the Columbia River Basin, is that of constructing artificial spawning grounds at favorable locations with the purpose of inducing natural

salmon spawning. This method has been used at one or two locations in British Columbia, and is now being investigated by the Washington State Department of Fisheries for a possible location immediately below McNary Dam. The procedure would be to construct gravel beds of proper gradation and slopes into which river water would be diverted to create conditions as near as possible to those in which salmon spawn naturally on the streams.

Effect of Reservoir Releases on Downstream River Spawning.—A development that may be of great future significance is the use of storage releases from reservoirs to improve stream conditions for salmon spawning in the river downstream. In the case of the Shasta Reservoir on Sacramento River it was found that the releases of stored water from the reservoir lowered the temperatures in the Sacramento River downstream by an average of about 20 degrees Fahrenheit during the summertime, as compared to natural flow conditions. This lower temperature, together with increased regulated low water flow, has proven very favorable to salmon spawning. Since the reservoir was placed in operation in 1942 the salmon spawning in the Sacramento below the dam has increased substantially over what they were prior to construction of the dam. This increase is said to have more than offset the losses caused by the blocking of upper river spawning areas by this high dam. A similar development is believed to be taking place to some degree on the Willamette River in Oregon below the storage reservoirs constructed on its tributaries. Especially favorable conditions appear to exist below Detroit Dam on North Santiam River where water temperature as released from the reservoir is about 45 degrees Fahrenheit. Relatively uniform releases are made during the summer and fall low flow period in the interests of power production at site, and to provide increased flow for navigation on the main stem of Willamette. A secondary benefit from this increased flow is reduction of the very heavy stream pollution that previously existed along the lower river. This factor again is of decided advantage to salmon migrations as well as to spawning, eliminating lethal water conditions that previously existed.

Summary of Achievements to Date.—Columbia River salmon and steelhead trout runs, which over a period of 70 years yielded an annual commercial harvest averaging nearly 400,000 cases, have in most recent years declined to the point where the annual commercial catch is not much more than 40 percent of the former amount. Sports fishing, on the other hand, has progressively increased over the years, and is now estimated to amount to the equivalent of at least 100,000 fish or 30,000 cases. Causes of the progressive decline are not clearly defined, but probably include stream pollution, effects of logging of watersheds, unscreened irrigation diversions, unladdered or ineffectively laddered dams cutting off spawning areas, reduction in summer flows because of irrigation diversions, ocean catch of immature salmon, ocean catch of sardines or pilchards—a salmon food source, and over fishing. The latter factor, while disputed by many, seems to be borne out by the fact that nearly comparable declines have taken place on streams or areas where none of the other factors are present.

Rapid development of the great power and other resources of the Columbia in recent years has brought into sharp focus the problem of maintaining the salmon runs. Whatever share of blame the dam builders have deserved in the past, all are now united in the determination that the fishery resource must be preserved, and engineering and biologists alike are working cooperatively to this end. Already many outstanding achievements have been recorded, and

a vast research program is under way. Physical accomplishments are summarized as follows:

a. Legislative actions to restrict fishing gear, places and time of commercial fishing.

b. Legislative actions to eliminate stream pollution.

c. Screening of canal intakes to prevent entry of fingerlings.

d. Laddering of dams at critical locations where none or ineffective ladders previously existed.

e. Removal or laddering of natural obstructions in rivers.

f. Establishment of a fish sanctuary program on lower Columbia River to build up runs in replacement of those lost upstream. This program is currently (1957) in the process of being extended to certain areas of the upper tributaries.

g. Inauguration on a large scale of artificial propagation by construction of hatcheries.

h. Use of hatcheries to sustain existing runs and to replace runs on rivers blocked by high dams.

i. Rehabilitation and enlargement of natural runs where practical.

Current research programs are being conducted on a cooperative basis, participated in by the Corps of Engineers and other dam-building agencies and by Federal and State fishery agencies, directed toward developing:

a. Better data on size and distribution of fish runs.

b. Improved means for upstream and downstream passage of fish over dams—(ladders, locks, elevators, tanks, barges).

c. Space and water supply requirements for fish passing facilities.

d. Methods of attracting and controlling movements of fish at dams to direct them to safe passages, with special emphasis of directional guidance of fingerlings.

e. Experimental investigations of fingerling passage through turbines and over spillways.

f. Experimental development of artificial spawning beds in or adjacent to streams.

Tangible accomplishments of the research program of practical significance to fish passage over dams and related problems include to date:

a. Demonstration that artificial propagation of several species of salmon can be successful, even to the extent of establishing new runs where none existed before.

b. Demonstration that adult fish, with proper attraction conditions provided, can be lifted over dams of any height by fish ladders, locks, or in tanks over tramways.

c. Demonstration that fingerlings can survive a free fall into water from any height, such as over a ski-jump spillway, thus providing a long step towards downstream passage of fingerlings over a dam of any height.

d. Indications that fingerlings travel generally near the surface of a reservoir, hence can probably be kept out of deep turbine intakes and thus simplifying the problem of attracting or guiding them to a safe exit over or through the dam.

e. Demonstration, on Willamette and Sacramento Rivers, that cold water releases from a reservoir may establish conditions favorable to spawning in the river downstream from the dam.

f. Development of the vertical slot type ladder.

DAM AND RESERVOIR DEVELOPMENT PROGRAM

What of the dams and generating plants? What influence has the fishery had on location, size and design of the structures? It has always been considered that high head type turbines would be lethal to downbound fingerling seeking passage through the power units. Experiments with Francis turbines at the 100-foot high Elwha Dam and under 184 feet at Glines Canyon Dam seem to partially refute this. On a large river such as the Columbia Kaplan type turbines are generally assumed to have distinct advantages to fingerling passage because of their large waterway openings, but this is true only under certain conditions. On the large Columbia-Snake River navigation dams this type was selected primarily because of the over-all higher efficiency over a wide range of head, and the economically desirable larger individual units that are physically possible of construction. In physical dimensions, such as wheel diameter, these units are from about 50 percent larger to twice as large as the largest competitive Francis wheels for the same head conditions. The larger Kaplans are accordingly slower moving in terms of r.p.m., and have a large gross area for water passage. These factors constitute to some degree an advantage to fingerling passage as applied to the Columbia and lower Snake installations. The limiting head of 90 feet to 110 feet used on these large installations has been adopted on the basis of economic factors and design considerations such as limiting load on thrust bearings. Whether or not this head limitation is also desirable from the standpoint of fish passage, as has been assumed by many, is not known and experimentation along this line is now planned.

As for location, almost any proposed dam on a fishery stream is subject to considerable criticism but again, recognizing the Salmon River spawning areas as vital to the fishery, it has been considered undesirable to place any major structure on that stream or to construct a high dam in the Snake River Canyon below the mouth of the Salmon and Imnaha rivers.

Design for dams, other than for fish facilities, has been influenced by the fishery primarily in the spillway studies. At run-of-river dams, the major migration of downstream fingerlings is during the annual period of spill. A spillway bucket with as little turbulence as practical and streamlined flow is most desirable. This is, of course, the opposite of the desired hydraulic effect for proper energy dissipation. Compromises have had to be considered and worked out by model studies in order to have spillway buckets as efficient hydraulically as practical under average conditions, and in the interest of fish life maintaining as high velocities with minimum turbulence as can be accommodated by downstream conditions.

CURRENT STATUS OF RIVER DEVELOPMENT

What of the river developments that have taken place or are under way since the advent of the "308" Report in 1948? Chief Joseph Dam, on Columbia River above the mouth of the Okanogan River, has been nearly completed and is in operation. The Dalles Dam, at the Dalles, Oregon, is under construction and final closure was completed this past winter. Ice Harbor Dam, the first of the four authorized dams on lower Snake River, is under construction. (See map for locations.) Farther upstream, in the Hells Canyon reach of Snake River, the Idaho Power Company is constructing Brownlee Dam in lieu

of the original high Hells Canyon project and hold FPC licenses for the Oxbow and low Hells Canyon dams. Downstream therefrom, preliminary planning of the Mountain Sheep and Pleasant Valley projects has been completed by the Pacific Northwest Power Company, although these projects have not received final FPC action for licenses. A low Priest Rapids Dam on the Columbia River, above the mouth of the Yakima, is under contract by the Grant County Public Utility District and work is now started. Wanapum Dam, next above and immediately below Rock Island, is in the planning stage. The Rocky Reach and Wells Dams on the Columbia River between the Wenatchee and Okanogan Rivers are under active planning by public utility districts and construction is being initiated this year (1957). The Corps of Engineers is doing preconstruction planning for the authorized John Day Dam on the Columbia River at the mouth of the John Day River.

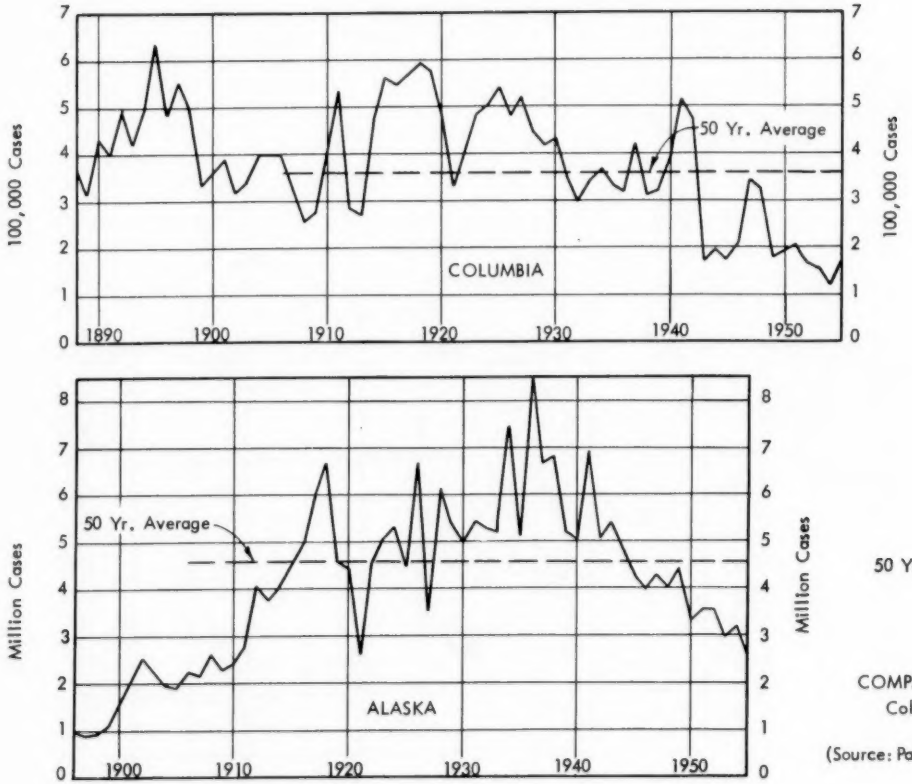
CONCLUSION

The intense interest indicated by this entire construction program attests to the need for regional development. The major monetary benefit from all of these projects is power production, with navigation also involved at The Dalles, John Day and Ice Harbor, and flood control features included at John Day, Pleasant Valley and Brownlee. There is vital need for electric power to meet the rapidly growing demand in the Pacific Northwest, where natural fuels are scarce. If these projects are all realized, it is evident that the obstructions and modified passage conditions that man has placed in the stream up to the present time are only minor in comparison to those which will confront the salmon on their migration and spawning routes 10 years from now. Man's ingenuity and the inherent resilience and stamina of the Pacific Salmon will be heavily taxed to assure the coexistence of these two great resources. Success will be based on sincerity and persistence in finding solutions to the problems of handling salmon. Where apathy was once quite evident when the subject of adequate care for migrating fish was considered, careful study and desire to protect the fishery are now evident. The commercial fishery recognizes that it has a responsibility for the resource. The Federal agencies, power companies and public utilities districts are expending large sums for fish passage facilities and for research, and have all engaged competent biologists and engineers to advise them on adequate conditions as well as engaging the State agencies to do specific research work for them. The Corps of Engineers is continuing its research program, model studies of construction problems, and improving the designs of fishways based on experience of the past.

The Pacific Northwest has been blessed with a tremendous resource, its falling water. To compensate for shortages in some other natural products, and with current population growths and industrial development needs, it is essential that this great water resource be conserved and used to its optimum potentialities. Among these potentialities are the salmon, a great natural resource furnished without cost by nature. It is of vital importance to the region that every effort be made to preserve this resource along with the development of others, and efforts to this end must continue even beyond the period of development that is now taking place.

The increased tempo of development and changed conditions that have become evident in the past decade since the "308" Report of 1948 have indicated

that that report should again be reviewed to plot the new development trends and needs for the next two decades. This second review is now under way and it is evident that additional projects will no doubt be investigated, authorized, and constructed that will further modify the action of the salmon as he proceeds upstream to his native area. Parallel with these investigations are additional research and physical studies to keep the salmon resource abreast of other resource developments.



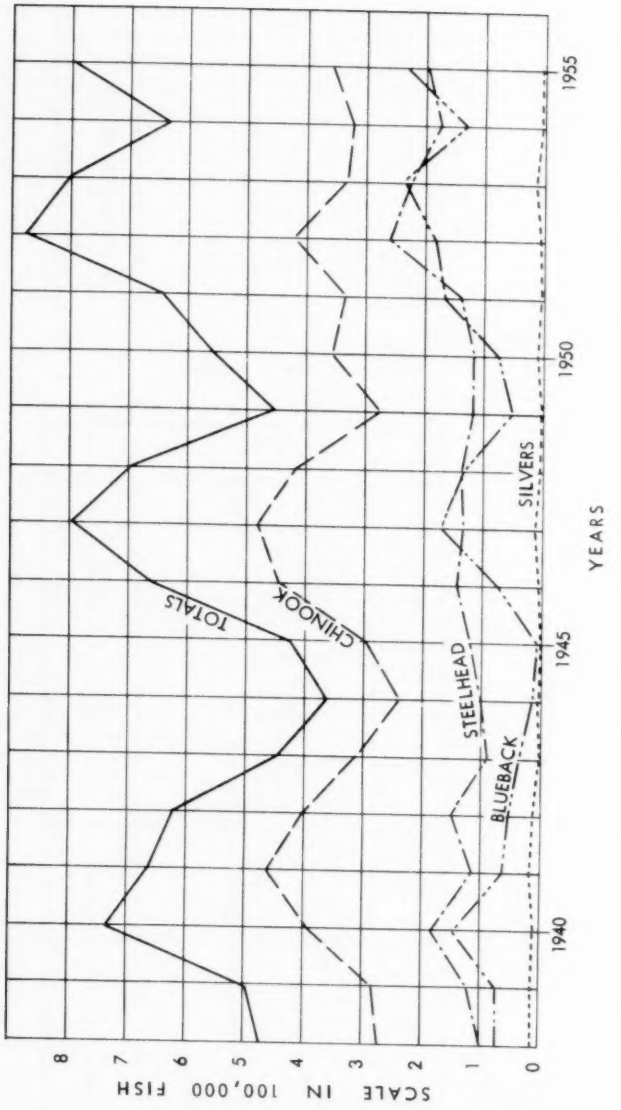
68 Yr. Average=377,000
50 Yr. Average=361,000

50 Yr. Average=4,580,000

COMPARATIVE SALMON PACKS
Columbia River and Alaska

(Source: Pacific Fisherman, Jan. 25, 1956)

Chart 1



RECORD OF SALMON & STEELHEAD
PASSING BONNEVILLE DAM
YEARLY TOTALS

Chart 2

COLUMBIA RIVER BASIN
U S ARMY ENGINEERS
SCALE IN MILES
25 0 25 50 75 100 125 150
LEGEND
PROJECT EXISTING OR UNDER CONSTRUCTION
PROPOSED PROJECT
CANADA
UNITED STATES
PACIFIC OCEAN
VANCOUVER ISLAND
VANCOUVER
VICTORIA
SEATTLE
TACOMA
OLYMPIA
ASTORIA
PORTLAND
SALEM
EUGENE
ROSEBURG
COOS BAY
MEDFORD
KLAMATH FALLS
CALIFORNIA
OREGON
NEVADA
IDAHO
UTAH
MONTANA
WYOMING
COLUMBIA RIVER
SNAKE RIVER
CLARK FORK
KOOTENAI RIVER
FRASER
THOMPSON R.
SKAGIT R.
YAKIMA R.
WILLAMETTE R.
ROGUE R.
UMPQUA R.
SNAKE R.
SALMON RIVER
CLEARWATER R.
GRANDE RONDE R.
JOHN DAY R.
DESCHUTES R.
PAYETTE R.
BOISE R.
OWYHEE R.
BOISE
IDAHO FALLS
POCATELLO
TWIN FALLS
LUCKY PEAK
AMERICAN FALLS
PALISADES
YELLOWSTONE NAT'L PARK
MISSOULA
BUTTE
LEWISTON
BRUCES EDDY
CASCADE
OXBOW
BROWNLEE
HELLS CANYON
SHEEP PLEASANT VALLEY
MCNARY
WALLA WALLA
PASCO
PRIEST RAPIDS
ICE HARBOR
WANAPUM
ROCK ISLAND
WENATCHEE
ROCKY REACH
CHIEF JOSEPH
GRAND COULEE
WELLS
SPOKANE
ALBENI FALLS
LIBBY
HUNGRY HORSE
CASTLEGAR
REVELSTOKE CANYON
MICA
ARROW LAKE
COLUMBIA LAKE
KOOTENAY LAKE
OKANOGAN LAKE
LAKE CHELAN
PEND OREILLE LAKE
COEUR D'ALENE LAKE
FLATHEAD LAKE
THE DALLES
JOHN DAY
BONNEVILLE
LONGVIEW
KELSO
CHEHALIS
YAKIMA
TACOMA
OLYMPIA
SEATTLE
VANCOUVER
VICTORIA
SKAGIT
FRASER
THOMPSON
OKANOGAN
CHELAN
COLUMBIA
KOOTENAY
CLARK FORK
FLATHEAD
SNAKE
WASH
IDAHO
MONTANA
OREGON
WYOMING
N

Chart 3

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

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DISCUSSION
(Proc. Paper 1416)

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by R. T. Richards, E. T. Keck, and J. Junget (Closure) 1416-3

Hydroelectric Power in the Southeast, by C. P. Lindner and L. F. Johnson. (Proc. Paper 1086. Prior discussion: 1225. Discussion closed.)
by C. P. Lindner and L. F. Johnson (Closure) 1416-5

Water-Hammer Design Criteria, by John Parmakian. (Proc. Paper 1216. Prior discussion: 1346. Discussion closed.)
by Benjamin Donsky 1416-7

Penstock Design and Construction, by George Robert Latham. (Proc. Paper 1285. Prior discussion: none. Discussion open until November 1, 1957.)
by Richmond Pearson Hobson 1416-9
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Multi-Layer Penstocks and High Pressure Wyes, by Ewald Schmitz. (Proc. Paper 1344. Prior discussion: none. Discussion open until January 1, 1958.)
by Sylvester V. Williams 1416-23
by Fritz Heidinger 1416-24

Note: Paper 1416 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, PO 5, October, 1957.

Discussion of
"HYDRAULIC DESIGN OF THE SANDOW PUMPING PLANT"

by R. T. Richards, E. T. Keck, and J. Junget
(Proc. Paper 948)

R. T. RICHARDS,[1] A.M. ASCE, E. T. KECK,[2] and J. JUNGET.[3]—Mr. Kerr mentions the use of reverse flow through pumps to relieve surges following pump tripout. Deliberate reversal of flow is also a convenient means of backwashing trash racks. Such a procedure should be used with caution, however.

A considerable velocity can be achieved in a pipeline flowing in reverse. Cutting the water off too quickly may result in a severe positive surge not provided for in the pipeline design; such is the case at Sandow. The Sandow surge protection scheme is based on the assumption that all positive surges will be preceded by a negative surge which will open the slow-closing vacuum breakers. If the pump discharge valves should close abruptly against reverse flow a very high initial positive surge would result. The rotovalves may be opened by local control to permit reverse flow, but if power failure should occur the solonoid valve mechanism would rapidly close the rotovalves on the emergency closure cycle, completely beyond the control of the operator. It was therefore stated in the operating instructions that under no condition should the rotovalve be used for reversing the pipeline flow. It must be locked open and reverse flow controlled by the hand-operated gate valve.

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2. Vice-Pres., Texas Power and Light Co., Dallas, Tex.
3. Resident Engr., Sandow Power Plant, Aluminum Co. of America.

Discussion of
"HYDRO-ELECTRIC POWER IN THE SOUTHEAST"

by C. P. Lindner and L. F. Johnson
(Proc. Paper 1086)

C. P. LINDNER,[1] M. ASCE, and L. F. JOHNSON,[2] M. ASCE.—The writers are indebted to Messrs. Price and Swennes for ably supplementing the paper. They have shown that the planned hydro-electric capacity and energy in the southeast can be suitably utilized in the expected load of the area by the time the planned projects can be constructed. Before a project is recommended for authorization, the fact that an appropriate demand for its capacity and energy is expected to develop is ascertained from the Federal Power Commission. But reconfirmation from time to time is reassuring. The Regional Office in charge of Mr. Price, which regularly prepares load forecasts, furnishes this information for the southeast. In the absence of an expected load demand at a load factor that will economically justify hydro-electric power installation no recommendation can be made for the installation of power facilities. In the Piedmont and Mountain provinces this usually means, as pointed out in the paper, that no project either for power, flood control, navigation, or any combination thereof can be economically justified unless there could be credited to the project sufficient supplemental values for conservation of natural resources, recreation, minicipal water supply, national defense, and the development of industry which generally follows assurance of an adequate controlled water resource. Up to the present time, with the exception of recreational boating on navigation projects, these supplemental values have not been used in arriving at the benefit-cost ratio upon which the recommendation for or against authorization of construction generally is based.

In addition to furnishing information on the future utilization in the load, the Federal Power Commission provides the Corps of Engineers with estimates of the unit values of capacity and energy at proposed plants. Moreover, advice of the Commission is sought on the capacity to be installed at each plant. It can be seen that the determination of the evaluation factor in the economic equation for the hydro-electric facilities of a project is a joint effort of the Federal Power Commission and the Corps of Engineers.

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Discussion of
"WATER-HAMMER DESIGN CRITERIA"

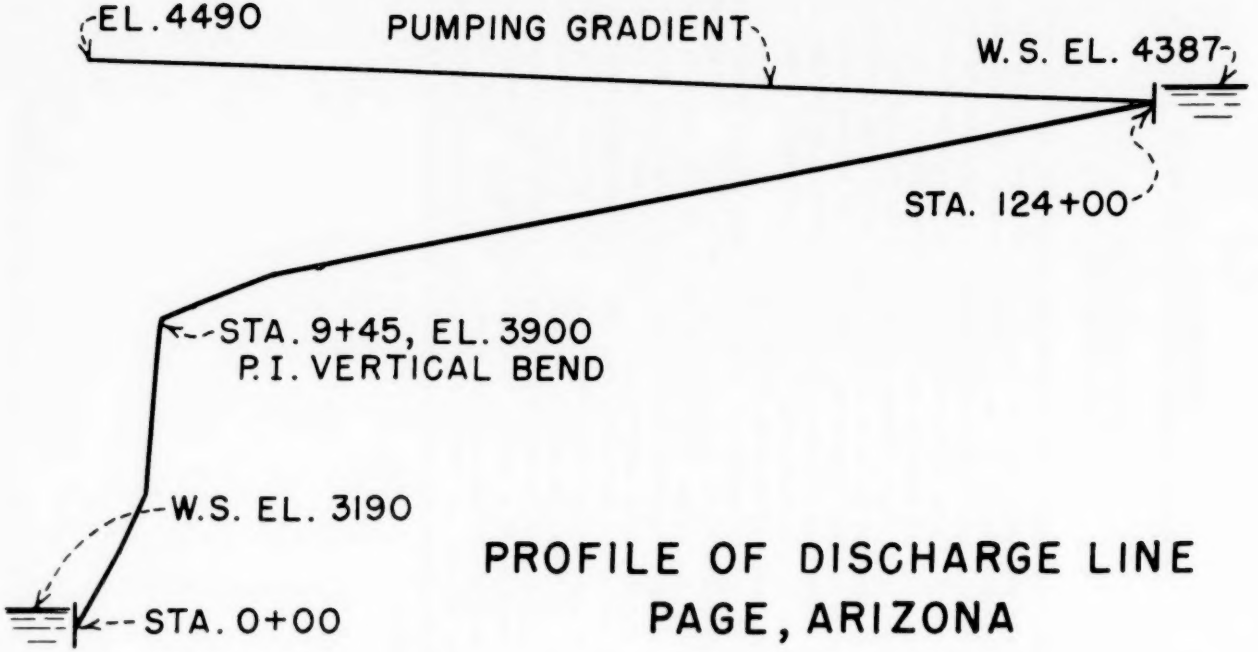
by John Parmakian
(Proc. Paper 1216)

BENJAMIN DONSKY.[1]—Mr. Parmakian has pointed out in his article that means should be taken to prevent the condition of water-column separation in pump discharge lines, except that it may be necessary for practical reasons to permit water-column separation to occur and to minimize the pressure rise due to the rejoining of the water columns by use of special devices.

Such a case occurs in the design of the pump discharge line in the domestic water supply for the Page, Arizona, facilities of the Colorado River Storage Project. As shown in the writer's figure, the profile of the discharge line is such that water-column separation could occur over a considerable length of the line if means were not taken to prevent it. Because of the high head, long line, and indefinite location of the water-column separation, it was not considered practical to prevent the separation.

It was, therefore, considered best to control the point of separation and limit the head rise due to rejoining of the water columns. This is done by the use of a surge suppressor at the pumps, and an air inlet and slow closing relief valve at the P. I. vertical bend (Station 9+45). The surge suppressor is designed to open quickly on interruption of power to the pumps and to discharge the full return flow of the line. Water-column separation will then occur at the air inlet and slow closing relief valve (Station 9+45), which opens quickly and remains open until the rejoining of the water columns. Due to the delay in closing the surge suppressor, and the steep profile between the surge suppressor and the P. I. vertical bend, the rejoining of the water columns will occur at the open air inlet and slow closing relief valve. This relief valve is designed to take the full return flow of the line at a predetermined head. By the use of this device the pressure rise due to rejoining of the water columns is minimized.

1. Engr., Technical Eng. Analysis Section, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.



PROFILE OF DISCHARGE LINE
PAGE, ARIZONA

Discussion of
"PENSTOCK DESIGN AND CONSTRUCTION"

By George Robert Latham
(Proc. Paper 1285)

RICHMOND PEARSON HOBSON,[1] M. ASCE.—Mr. Latham's paper is a concise presentation of factual information derived from many years of successful experience by him and the company with which he is associated. The examples have been well chosen to illustrate several installations involving different applications of design and construction.

The writer has prepared a discussion which briefly outlines some practices of the Army Corps of Engineers in the design and construction of penstocks. The facts contained herein are offered solely with the idea that they may add something to the general pool of information on the subject of penstocks, and are not intended as a solution to all of the many problems arising in the design, fabrication and erection of these structures.

With the advent of welding immediately following World War II, general criteria for the design and fabrication of penstocks were adopted. These follow, for the most part, the "Rules for the Construction of Unfired Pressure Vessels" of the ASME Boiler Construction Code, but are not restricted entirely to the provisions of that Code. For instance, tangential working stress is taken as one-half the minimum yield strength of the material rather than one-quarter of the ultimate strength. The rules for joint efficiency, however, are observed and may be restated as follows:

a) Without stress relief or radiographic inspection 80%
b) With stress relief, without radiographic inspection 85%
c) With radiographic inspection, without stress relief 90%
d) With both radiographic inspection and stress relief 95%

Oven stress relieving is specified only where needed to relieve residuals and to prevent weld cracking. It is not considered economical to use it in order to gain an additional 5 percent joint efficiency. On the other hand the difference in cost between complete radiographic inspection and spot examination of longitudinal welded seams will usually be offset by the metal saved through an increase of 10 percent in joint efficiency.

Penstocks are usually proof-tested to 150 percent of the design head, including pressure rise. This serves, not only as a test, but provides, mechanically, a certain amount of stress relief, especially of a highly local nature. Proof-testing is usually accomplished by provision of bulkheads at the ends of the pipe and at the surge tank riser, if any. The penstock is then filled with water and pressure applied by means of pumping. The desired pressure is usually applied and relieved three times, being held the last time for a period sufficient to make a thorough inspection. While it has not always been the case, it is now the practice of the Corps to specify a minimum water temperature of 60°F during testing. This will in some cases require heating of the water.

1. Engr., Office of Chief of Engrs., U. S. Dept. of the Army, Washington, D. C.

In the past it has been the practice of the Corps of Engineers to require the interior surface of penstocks to be coated with hot applied coal tar enamel. This choice has been based upon long-term successful experience, not only by the Corps, but by others. The difficulty of application, however, plus a destructive fire, has led to the consideration of the use of vinyl type paints. For one installation, not yet built, it has been estimated that a vinyl paint system having one-half the useful service life of coal tar enamel would still be the more economical choice, even when taking into account the cost of plant shut-down and loss of power revenue during repainting. While, at present, there is no assurance of the expected life of vinyl in penstocks, it has been used in one large installation, being justified on the basis of a well calculated risk.

Most of the penstocks designed and built by the Corps have been fabricated from plates conforming to ASTM-A-285, Grade C, Firebox Quality. This material has a minimum yield strength of 30,000 psi which allows a working stress of 15,000 psi prior to application of the joint efficiency factor. Consideration is now being given to the use of steel conforming to ASTM-A-201, Grade B. The greater minimum yield strength of the latter steel (32,000 psi) will allow a reduction in tonnage of between 6 and 7 percent. This applied to an installed price of 25 cents or more per lb., will usually pay the difference in premium with the net result, that the steel with generally greater notch toughness and lower transition temperature may be secured at no extra cost.

The following examples are offered as typical of the several installations designed and built by the Corps:

Detroit Dam is located on the North Santiam River in Oregon. (Fig. 1) Water is delivered to the power turbines through two 15-foot diameter penstocks. The water head at the center of intake is 172 feet and the head on the lower portion 380 feet. The penstocks are designed for maximum head, with plate thickness varying from 11/16 inches at the top to 1-3/16 inches at the bottom. The exposed portion of these penstocks have stiffener rings at about 5-foot spacing. (Fig. 2) It may be noted that during the period that these penstocks were designed, all penstocks, designed by the Corps, were provided with stiffeners whether or not they were subject to external loading. Present-day practice has eliminated stiffening on unencased penstocks unless they are needed for some specific purpose.

Blakely Mountain Power Plant is on the Ouachita River in Arkansas. The penstock installation consists of a 24-foot steel power conduit in a 30-foot diversion tunnel about 760 feet long to a wye branch just downstream from the surge tank. (Fig. 3) At the wye it branches into two 16-foot penstocks. The large conduit is supported in the tunnel by pedestals at 40 feet on center. The moments and shears set up by the concentrated reactions are resisted by ring girders at the support. The design of the intermediate shell and the ring girders conforms to the principles outlined in paper 1829, Trans. ASCE, 1931, "Design of Large Pipe Lines," by Herman Schorer. The decision to construct a free-standing penstock rather than a lined tunnel was based upon economics, since the 30-foot tunnel was needed for diversion and the cost of a three-foot ring of concrete to reduce it to 24 feet was greater than that for a free standing structure. The wye branch and the riser are encased within the surge tank base.

Fort Randall Dam is on the Missouri River in South Dakota. (Fig. 4) The installed power consists of 8 units of 40,000 KW each, or a total of 320,000 KW. The penstocks are 22 feet in diameter and are supported at 24-foot intervals

by pedestals and ring girders. (Fig. 5) The static design head was 145 feet with a pressure rise varying as a straight line from zero at the intake to 62 feet at the turbine. The decision to erect these penstocks as free-standing within the tunnel was based upon the notoriously erratic foundation conditions in that part of the Missouri River Valley. Unequal settlements and rebounds of such magnitude were expected that it was not, at that time, deemed safe to encase the penstock in concrete, but to keep it accessible for inspection and for shimming of supports against unequal settlement. (Fig. 6)

100% radiographic inspection was specified on this job. This proved to be a good investment, in that slag inclusions, porosity and incomplete penetrations were disclosed to be somewhat prevalent in the early stages. To cut down the repair work and to make better progress, the contractor instituted a school to improve the welding technique of his men. Improvement was soon apparent and repair work considerably decreased.

At Fort Randall carefully controlled methods for handling a penstock section from the storage yard to the tunnel, permitted the shop application of coal tar enamel except for a traffic strip in the invert and a band at each end. This resulted in expedition of the work and the minimizing of a potential fire hazard.

Another innovation, insofar as pertains to penstocks built by the Corps, was the sealing of the turbine wicket gates and elimination of the downstream hemispherical bulkhead required for proof-testing.

Observations, since the completion of this installation, indicate that greater advantage should have been taken of beam action and foundation capacity to provide a greater distance between supports. In addition to greater economy, the increased flexibility of longer spans would make the penstock less sensitive to differential settlement.

Chief Joseph Dam and Powerhouse on the Columbia River in Washington has an initial installed capacity of 16 units at 64,000 KW, or a total of 1,024,000 KW. Provision has been made for the ultimate installation of 11 more units, bringing the total ultimate capacity to 1,728,000 KW. The average operating head is 175 feet with a design head on the lower sections of penstocks of 200 feet. The penstocks are 25 feet in diameter and were designed without stiffeners except in the encased portion. (Fig. 7) Each penstock is supported about half-way down the slope with a ring-girder type of pedestal and passes through a cut-off wall just upstream from the bottom elbow anchorage.

Garrison Powerhouse has an installed capacity of 400,000 KW derived from five units of 80,000 KW each, three of which have been completed, while the other two are still under construction. (Fig. 8) The five penstocks are 24 feet in diameter and about 1600 feet long. They are supported on 180-degree saddles at 24-foot centers. The upstream portion of about 400 feet is provided with articulated joints at 48-foot centers spaced midway between saddles. (Fig. 9) The penstocks in this region are stiffened adjacent to the saddle faces in order to minimize distortion of the shell at the articulated joints. The next 800 ft. of penstock consists of a plain pipe shell passing continuously over saddle supports without stiffeners. Articulated joints are provided thence to the tunnel portals. A 44-foot section supported by ring girders on rocker pedestals spans the joint between the tunnel portal and surge tank support structure. A 129 ft. section, supported near the ends by ring girders, spans the area between the surge tank base and the powerhouse.

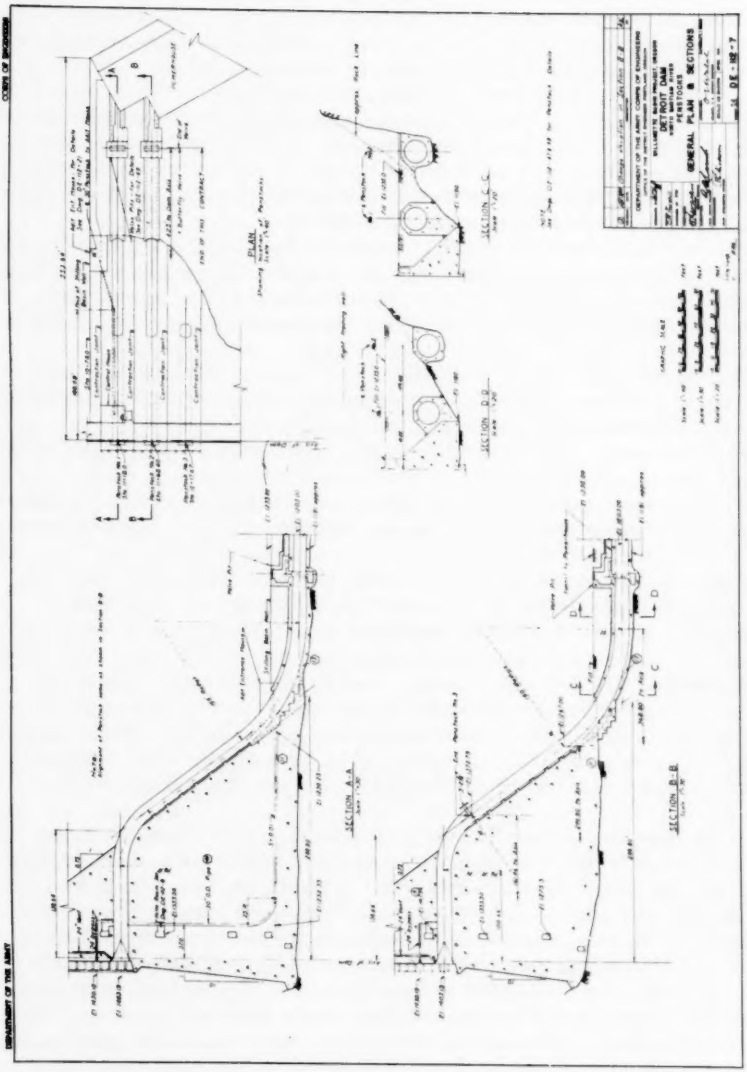


FIG. 1 - DETROIT DAM - PENSTOCKS

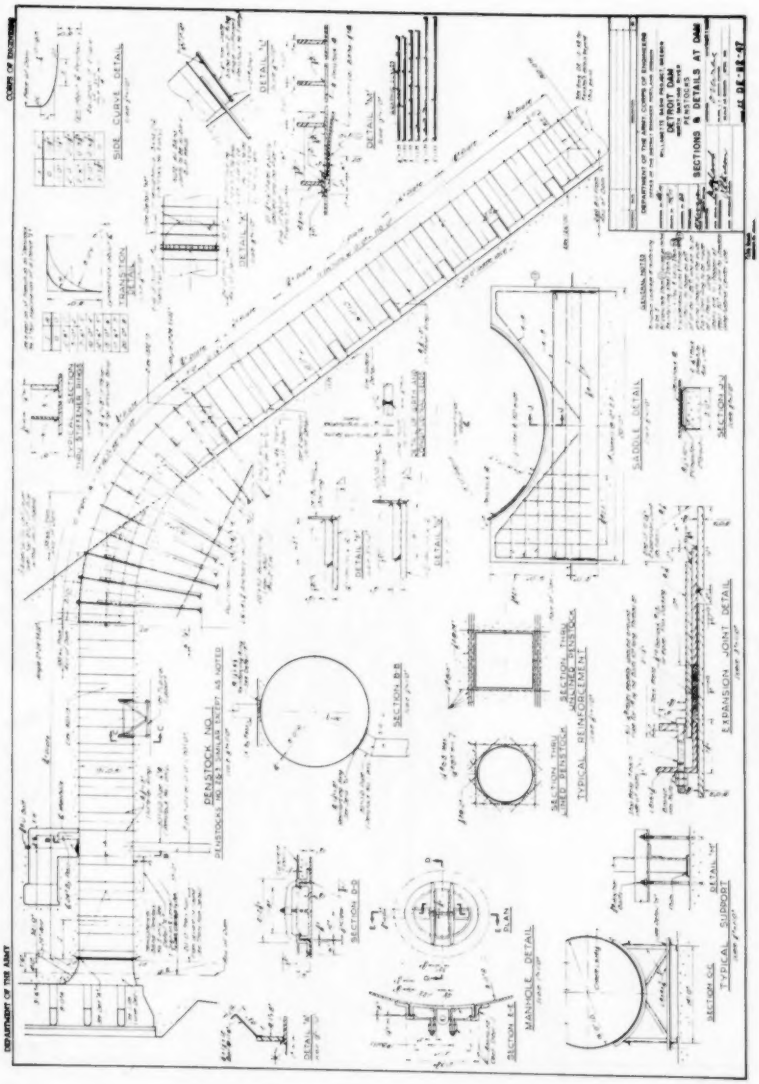


FIG. 2 - DETROIT PENSTOCK DETAILS

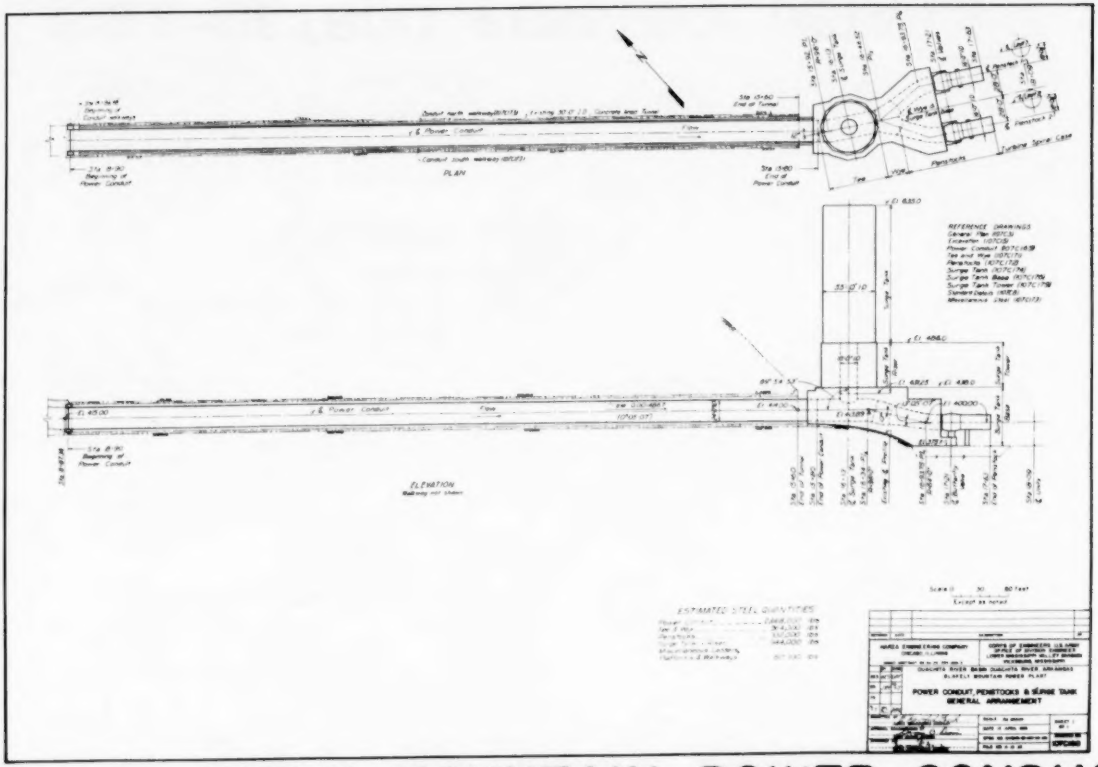


FIG.3-BLAKELY MOUNTAIN POWER CONDUIT

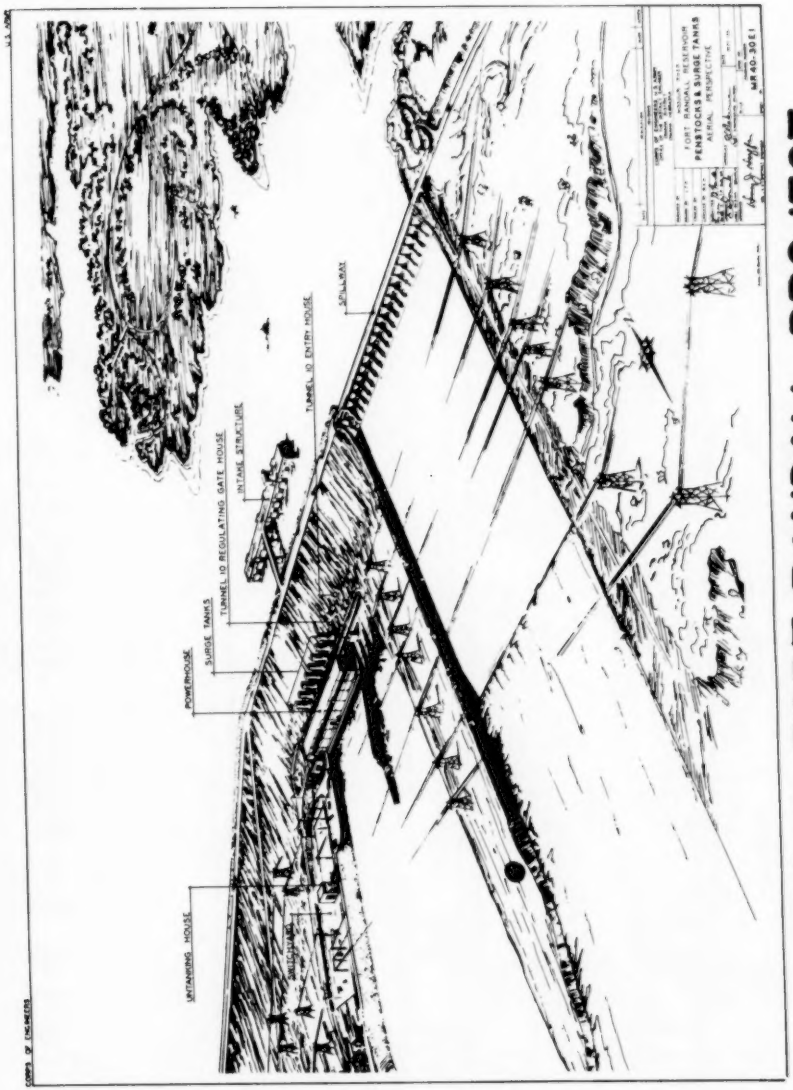


FIG. 4 - FORT RANDALL PROJECT

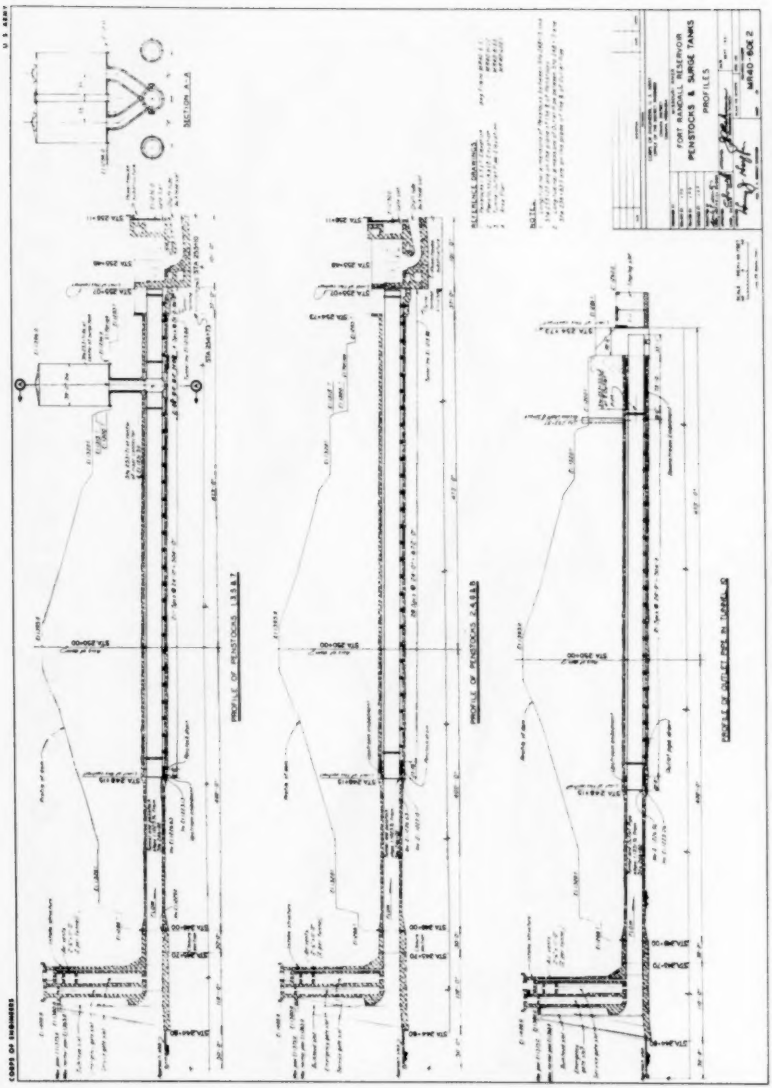


FIG. 5 - FORT RANDALL PENSTOCKS

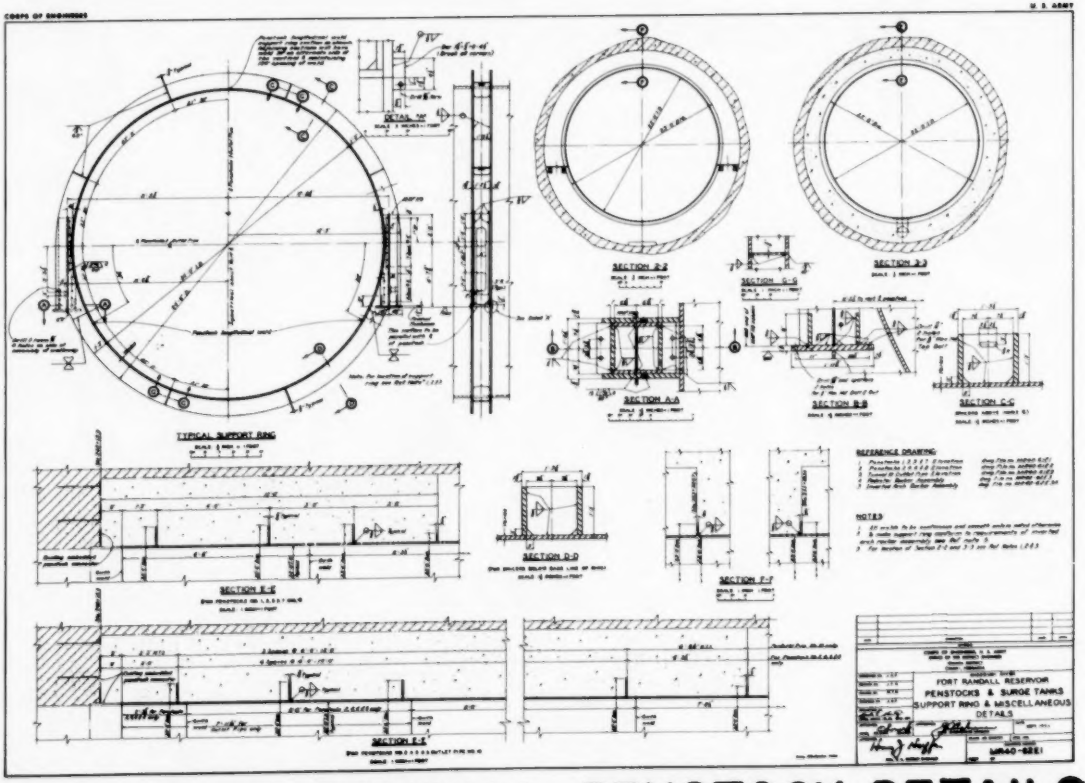


FIG. 6 - FORT RANDALL PENSTOCK DETAILS

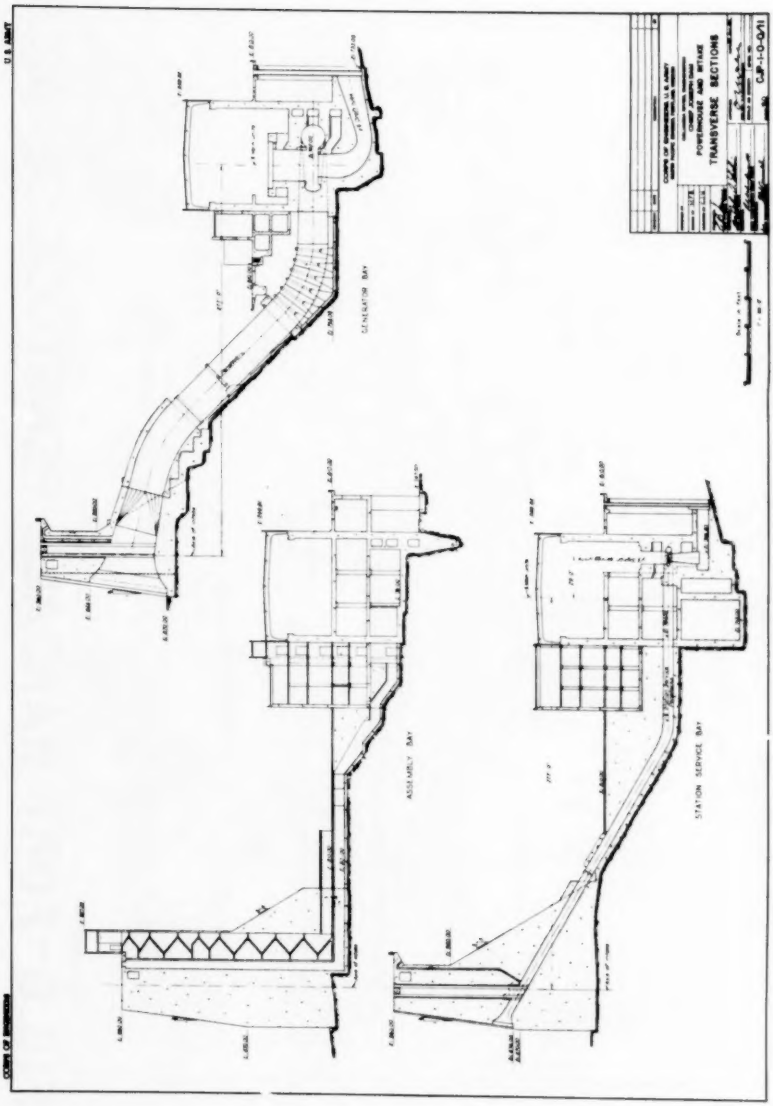


FIG. 7 – CHIEF JOSEPH POWER INTAKE

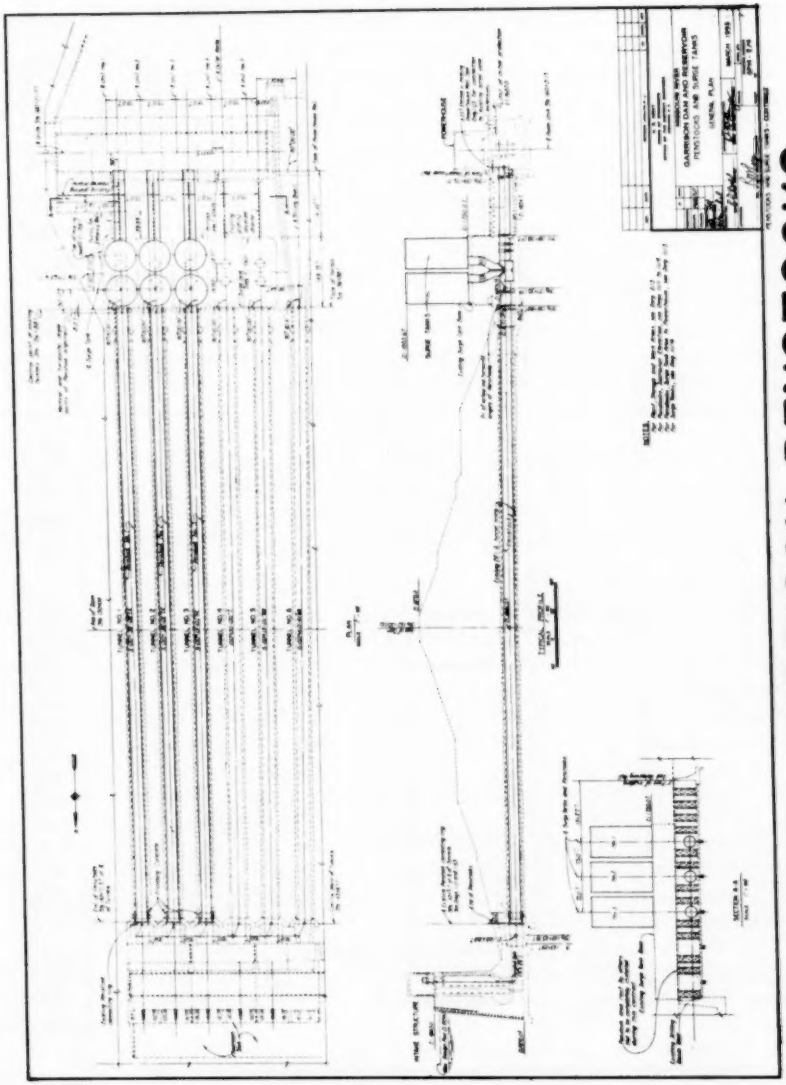


FIG. 8 - GARRISON PENSTOCKS

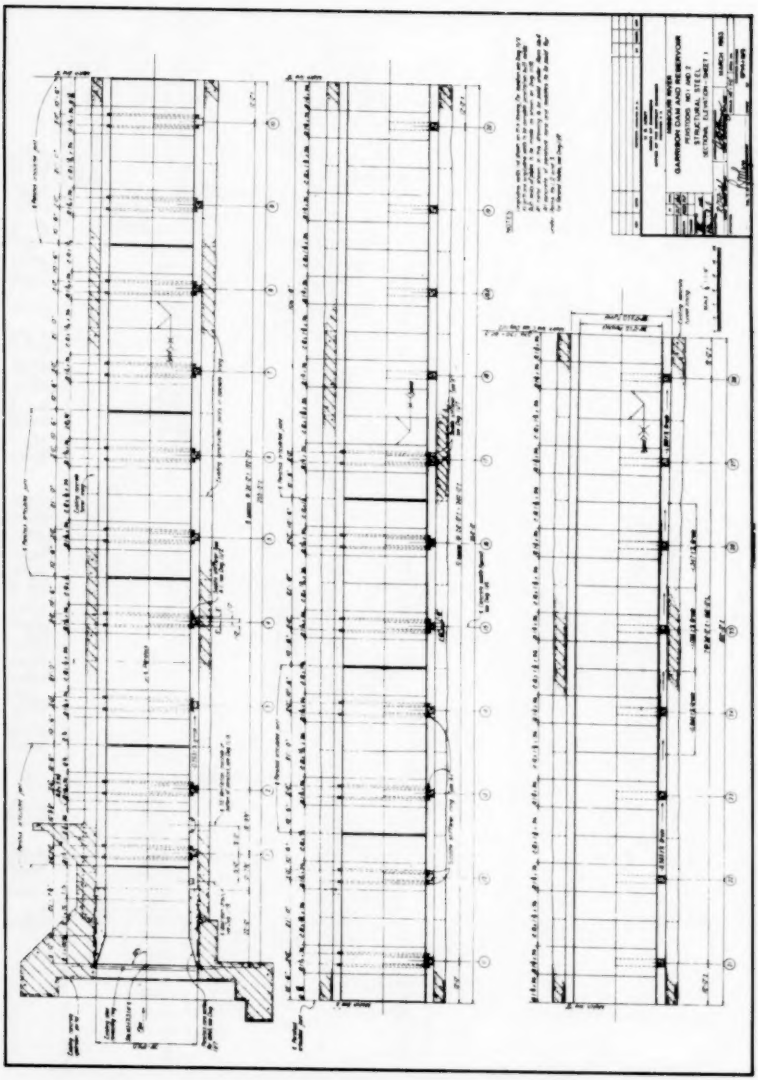


FIG. 9-GARRISON PENSTOCK DETAILS

The expected rebounds caused by settlements induced by the weight of reservoir water at one end and the powerhouse at the other end, together with the erratic behavior of the Fort Union shale in the foundation, not only dictated free standing penstocks but also that they be articulated over a portion of their length. The articulated joints are unusual in that a joint of the Dresser type had not been used on penstocks of this size before. This type of coupling was specified because it was believed that the standard packing gland type did not provide enough flexibility for the expected movements. Furthermore, the large number required indicated that the cost of conventional joints would be prohibitive. It was estimated that the latter would cost at least 5 times as much as the Dresser type.

That portion of the penstock shell immediately adjacent to the horns of the saddle is subject to high localized bending stresses of somewhere near the yield point during filling and emptying. If the yield strength is exceeded, relieving will occur in the nature of plastic yield hinges and the upper portion of the plate will function as a two-hinged arch. The design of unstiffened penstocks on saddles was motivated by a paper by Mr. L. P. Zick of the Chicago Bridge and Iron Company on the subject of horizontal tanks, on saddle supports. This paper appeared in the Welding Journal Research Supplement in September 1951. This installation is perhaps an indication of the more liberal approach to penstock design as compared with the overly-stiffened earlier designs.

In conclusion, it is believed that the design and construction of penstocks, while basically an interesting subject, provides the greatest interest to the engineer in the attempt to solve the many problems imposed by local, rather than general, conditions.

FRITZ HEIDINGER.[1]—The author's paper covers a wide range of situations encountered by his company and is a valuable contribution.

At the beginning of his paper the author states the basic assumptions selected by his company for the design of penstocks. It is well known that the design criteria used by various organizations differ considerably as there are no acknowledged standards of practice for penstock design. The following comparison of the author's criteria with those used by the U. S. Bureau of Reclamation shows a substantial difference in the design stresses used and poses the question as to what the basis for a safe design should be.

Inasmuch as a penstock is a pressure vessel, the Bureau follows the rules of the ASME Code for the construction of Unfired Pressure Vessels as far as they are applicable. The differences between the requirements of this code and the author's criteria are considerable. For the A285 Grade C steel, the basic code design stress is 13750 psi and the allowable joint efficiency for spot-radiographed joints is 80 percent. On this basis the net design stress used by the author is 39 percent higher than the code stress. Obviously large savings are possible if this higher design stress can be used without impairing the safety.

In the construction of penstocks the Bureau uses only materials permitted by the code. The A285 steel in Grades B and C has been used extensively in the past. As the transition temperature of this steel lies within the range of operating temperatures, it is possible that brittle fractures develop,

1. Penstocks and Steel Pipe Section, Div. of Design, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

particularly in heavy plates. This was forcibly brought to our attention some years ago when a 15-foot diameter penstock was severely damaged by a brittle fracture occurring during a field pressure test at a temperature of approximately 40°F. In order to minimize the danger of brittle fractures, we now generally specify A201 Grade B steel where plate thicknesses exceed one inch because this steel has better characteristics in this respect.

It is noted that the author's organization does not customarily provide an overpressure field test. In our opinion this is the most desirable test provided the entire length of the penstock can be subjected to the full test head. In this case all parts of the penstock including welded field joints are tested and a hydrostatic shop test can be omitted which is particularly desirable for large diameter penstocks requiring special test equipment. However, for steeply inclined or buried penstocks a field overpressure test may not be practical.

Due to the advantages enumerated in the description of the Paucartambo penstock, Dresser-type couplings are used extensively in penstock constructions. We are now designing a 16-foot diameter penstock with plate thicknesses up to two inches which will have coupled field joints to avoid difficulties with the field welding and to eliminate the necessity for stress relieving of the field joints.

Three of the penstocks described in the paper are supported on concrete saddles. We do not use saddle supports for pipes larger than 8 foot in diameter. The friction forces on saddles are very high as compared with other types of supports. Unless the pipe is adequately stiffened close to or directly over the supports very high stresses occur at the horns of the saddles. It is known that large thin-shelled pipes without stiffeners often become dented at these points.

Very little useful information is available on the design of tunnel liners and the determination of the size of stiffener rings are buried penstocks. It would be of interest to know how the thickness of the liner for the Kerr Development was determined and what loading assumptions were used for calculating the size of the stiffener rings of the penstocks for the C. T. Strike and Pelton Developments.

Discussion of
"MULTI-LAYER PENSTOCKS AND HIGH PRESSURE WYES"

by Ewald Schmitz
(Proc. Paper 1344)

SYLVESTER V. WILLIAMS.[1]—The writer is to be congratulated on this excellent paper. It reveals that he has done much pioneer thinking and work in finding a better way to utilize the inherent stabilized energy potentially available in thin plate rolled from commercially lower carbon steel ingots. The multi-layer construction of the walls of the penstock and wyes fosters this utilization. Man's ability to better utilize the energy about him has enabled him to shift the physical burden from his own back, and thus have more freedom. This search for a better way has now reached its highest level in man's desire for a better life. In our highly competitive changing world, this search must be continued if we are to remain free. In other words, to lift the burdens from the backs of all the world's people, requires that we use all the energy available to us, in the best way possible.

This paper indicates that the author's corporation has done much research in developing the multi-layer construction, and that this way of better utilizing the inherent energy to be found in the thinner plate steel is helping the people of South America, as well as our own people, in their upward struggle.

The basic advantage of multi-layer construction is naturally well known to the writer, but to many professional engineers it is still new. Many are not aware of all of the differences in the properties of thin and thick steel plate of the same chemical analysis. The recent investigations on ship plate clearly point this out. The conclusion of this investigation was to change the chemical analysis on plates thicker than 1/2". This paper offers another solution with better utilization of the energy of the material, with no increase in material either chemically or physically.

Mr. Schmitz has itemized five points on the advantages of thin plate over thick plate. Number 1 on "visual inspection after rolling at the steel mills" can readily be visualized. We believe in good inspection, and know that surface defects alone do not account for the difference in the properties in the thin plate from those of heavier plates. The inherent quality of the thin plate over the thicker plate can better be evaluated by a macro etch and micrographs on the cross section. Here you can better visualize the greater uniformity of the structure of cross section of thin versus thicker plate of the same heat of steel in the as-rolled condition. This becomes more noticeable with the thickness over 1/2" compared to 1/4" or under. This is caused by the slower cooling rate from rolling temperature on the heavier plate. Heat transfer, or energy loss, is slower at the higher temperatures, and faster at the lower temperature through the plate thickness. The action is exponential. Hence, grain size is more uniform from each surface on thin plates.

On point 2, the yield point and ultimate strength are normally higher on thin plate because of the additional working. However, the cooling rate and

1. Head, Dept. of Eng., University of Omaha, Omaha, Nebr.

practice on the hot bed can greatly affect this. If stacked shortly after the last pass of rolling, there would be little difference, except for the top and bottom plates on the stacked pile. This practice may account for the fact that the 1/2" Inner Shell of L-3 and L-4 section referred to on page 6, has a higher yield and ultimate than the 1/2" Inner Shell of L-1 and L-2, as well as the second Layer Half of 1/4" plate. In other words, controlled cooling from rolling temperature is of vital importance not only for the yield and ultimate, but more so for the susceptibility to brittle fracture. Unless this cooling is controlled correctly, the transition temperature of the thick plate can be just as low, if not lower, than the thin plate, and Item number 5, would be reversed.

As pointed out in Item 3, the ultimate strength of the shell will approach that of the steel. The certainty is still dependent upon how well the stress is distributed by the deformation of the inner shells to those of the outer shells. The outer layers are subjected to less deformation or unit strain. This can easily be visualized when there is more difference between the outside and inside diameters of any shell. When the operating pressure approaches the allowable design stress, the yield point of the layer material must be given consideration, if a factor of safety is to be known with certainty, as it must when operating with such high pressures.

It should be pointed out that the data on deformation on the burst test, Figure 3, is greatest on that section of the vessel having the lowest yield point material in its inner shell. The point of fracture indicates that the inner shell of L-1 and L-2 has functioned best in distributing the stress throughout its cross section, and that the fracture starting at C_3 may have been influenced by the inner shell of L-3 and L-4, setting up some additional stress by the unequal deformation rates. This is caused by the difference in yield point of steel of the inner layer. This is indicated by measurements taken at 3000, 3615, and 4260 psi. The bursting pressure of 79,200 is very close to that of the inner shell strength of L-1 and L-2, namely, 79,900. This indicates better utilization of energy in this plate than that on the plate of the inner shell of L-3 and L-4. The ratio on inner shells is 99.12 in L-1 and L-2, and 90.3% in L-3 and L-4. While 90% is good, the 99.12% is better, and this means that if 90.3% might be considered excellent, 99.12% would be perfect, and certainly the better way to build penstocks and wyes.

Again, congratulations are offered Mr. Schmitz on his very fine paper, and the good intentions that are in back of his motives of continually searching for a better way to fabricate penstocks and wyes, and utilizing energy.

F. HEIDINGER.[1]—In this very interesting paper the author gives a detailed description of the fabrication and installation procedures for multi-layer penstocks which are not generally known to engineers. The unusually heavy high pressure wyes described are of particular interest to penstock designers.

Considering the large number of multi-layer pressure vessels that have been built to date, this method of construction is well established. The Bureau of Reclamation has in operation a number of gate hoists with multi-layer cylinders 30-inches in diameter with 1-3/4-inch thick walls which were fabricated by A. O. Smith Corporation 16 years ago and have given satisfactory

1. Penstocks and Steel Pipe Section, Div. of Design, Bureau of Reclamation, Denver, Colo. U. S. Dept. of the Interior.

service. The bore of these cylinders had to be machined to very close tolerances.

Although the author states that multi-layer construction may offer economic advantages if the pipe wall thickness exceed 3/4-inch, it is doubtful whether it is competitive with solid-wall construction in the range of thicknesses usually required in penstocks. Solid-wall penstocks with plate thicknesses up to 2-inches are not uncommon. Even if a saving in weight can be claimed for multi-layer penstocks there is no compelling reason for using them unless the overall cost is lower than that of solid-wall penstocks.

The greatest difficulties in penstock construction occur in the fabrication of thick-walled fittings such as those described by the author, particularly where field assembly is necessary on account of limited shipping clearances. The Bureau has recently completed a penstock manifold similar to the one built for the Cubatao plant. The header of this manifold is 26 feet in diameter and has a wall thickness of 1-5/8-inch. There are four branches 14 feet in diameter and two branches with a diameter of 12 feet. Mushroom type reinforcements with webs 2-1/2 and 3-inches thick were used to reinforce the outlet openings. Due to the severe restraint produced by the heavy plates frequent cracking of the welds occurred during field assembly. A major factor in eliminating this difficulty was the insertion of an intermediate stress relief operation when the heavy welds were about half completed. We have never designed wye branches as heavy as those for the Kemano penstocks. It would be of considerable interest to know what welding procedure was used in fabricating these heavy pieces, whether they were preheated, and whether more than one stress relief operation was required.

We also designed a very large manifold for Glendo Powerplant on which we made tests which approximate very closely strain gage readings on model.

Paper 1419 October, 1957 PO 5

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

SIR ADAM BECK—NIAGARA GENERATING STATION NO. 2*

Frank Dobson[1]
(Proc. Paper 1419)

ABSTRACT

This paper presents the design and construction features of an extension to a hydroelectric power generating station on the Niagara River and its associated works, with particular emphasis on the pump-storage-generating plan used to preserve the scenic spectacle of Niagara Falls.

Construction started on the new Sir Adam Beck Niagara Generating Station No. 2 in the fall of 1950, immediately following the ratification of a new treaty between the United States and Canada permitting additional diversion of water from the Niagara River for power purposes. This new treaty stressed the preservation of the scenic spectacle of the falls by stipulating that during the daylight hours of the tourist season a minimum of 100,000 c.f.s. of water must flow over the falls; at all other times, a minimum of 50,000 c.f.s. The remaining river flow over these amounts could be equally divided between the United States and Canada for power purposes.

General Scheme

The Niagara River in flowing between Lake Erie and Lake Ontario drops 326 feet; 315 feet of this fall is concentrated on a seven mile reach in the river between the villages of Chippawa and Queenston, approximately 225 feet in the rapids above the falls and the falls themselves, and a further 90 feet in

Note: Discussion open until March 1, 1958. Paper 1419 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 5, October, 1957.

*Presented at a meeting of the American Society of Civil Engineers in Buffalo, N. Y., June, 1957.

1. Project Manager, Sir Adam Beck Generating Station No. 2, Niagara Falls, Ontario, Canada.

the lower rapids below the falls. The new Sir Adam Beck project is designed to utilize the full head concentration of this reach together with the maximum permissible diversions within economic limits.

The project consists of two submerged intakes, of the Johnson-Wahlman type, constructed along the bank of the river about 1/4 of a mile downstream from the village of Chippawa; twin tunnels driven at 51 feet diameter, lined with three feet of concrete for a finished diameter of 45 feet, 5-1/2 miles long beneath the city of Niagara Falls; an open cut canal some 2-1/4 miles long and 200 feet wide leading to the forebay of the power plant which is located on the lower river near the village of Queenston; a pumping generating station with associated reservoir located adjacent to the power canal; and remedial works in the upper river including a control structure below the intakes to regulate the level of the Grass Island pool.

Construction Progress

To date construction has been completed on the intake structures, the twin tunnels, the power canal, and for twelve units of the Beck No. 2 powerhouse. The first of these 105,000 h.p. units went into service in April 1954, No. 12 in August 1955, giving an installed capacity at the present time of 1,260,000 brake horsepower.

Presently under construction on the project is the Grass Island Dam control structure, the pump storage powerhouse, and the extension to the Beck No. 2 station for four more units.

Grass Island Dam

The Grass Island Control Dam is being constructed in accordance with the 1950 treaty to control the water level of the Chippawa Grass Island pool in which reach of the river are located the new intake structures for both the United States and Canadian power diversions. Without this dam the water in the pool would drop four feet with maximum diversion. This would virtually eliminate the flow of water over the American Falls and would also have the effect of lowering the level of Lake Erie. Furthermore, the dam provides means of regulating the flow over the falls within the minimum specified flows of 100,000 and 50,000 c.f.s. In effect it provides a means of turning the water off and on. (Fig. 1)

The completed control dam extends 1,550 feet out in the river from the Canadian shore and it contains 13 - 100 foot sluices. The flow through these sluices is controlled by steel gates hinged at the sills and opened or closed by hydraulically operated pistons located in the piers at each end of the gate. These gates are among the largest submersible type in the world, each 100 feet wide and 10-1/2 feet high above the sill elevation. The gates were supplied by the Canadian Vickers Company and installed by Ontario Hydro. As access to the piers there is a 24 foot wide service deck running the entire length of the dam. This roadway is carried between the piers on prestressed concrete girders. This was the first use of prestressed concrete on a large scale by Ontario Hydro.

The most unique construction feature of the Grass Island Dam was the steel crib type cofferdam designed by Ontario Hydro. This cofferdam is basically a series of steel cribs, 30 feet wide, 15 feet high, and 10 feet long, supported off the bottom of the river by 12 X 12 H pilings fitted into slots in

AERIAL VIEW OF GRASS ISLAND DAM CONTROL STRUCTURE. COFFERDAM IN PLACE AROUND THE LAST FOUR GATES.

Fig. 1

the steel frames, weighted down by precast concrete blocks, with the water sealed from the enclosure by steel sheet piling.

The construction of the Grass Island control structure which was started in January 1954, is virtually complete; all 13 gates have been installed and the remnants of cofferdam are being removed from around the last four gates. Clean-up work is in progress in the construction area which before this summer is ended will be landscaped and restored to its previous natural park land state.

The Grass Island control dam is part of the over-all plan of the Niagara River remedial works. Another phase of this plan included work at the Horseshoe Falls where excavations were completed on both the American and the Canadian flanks to more evenly distribute the flow over the full crest of the falls. The remedial works has been an international effort carried on and paid for jointly by Canadian and United States agencies. The Canadian agency being the Ontario Hydro and the American agency being the United States Corps of Engineers.

Pump Storage Reservoir and Powerhouse

The second division of the project still actively under construction is the pumping-generating station.

The pump storage scheme is to store water at night, during the hours of low power demand and when surplus energy is available, by pumping into a reservoir; this water to be used during the daylight hours of high demand, back through the same pumps reversed in direction to become turbines. After discharging from the pump storage plant the major advantage of the stored water is obtained by its re-use at either of the main generating stations where a high head is available. (Fig. 2)

There are three main components in the pump storage development. A canal "A" leading from the main open cut canal; a pumping generating station "B" and a reservoir, "C".

The canal is 1,450 feet long and 140 feet wide. The average rock cut was 58 feet, with a maximum at the powerhouse of over 80 feet; depth of overburden averaged 20 feet. This excavation has now been completed and the canal flooded.

Construction of the reservoir has been completed ready for filling. The reservoir covers an area of 750 acres and will provide 15,500 acre-feet live storage on the basis of a 25 foot drawdown. The reservoir is formed by an embankment approximately five miles long varying in height from 15 to 65 feet. The embankment is essentially a rock mass made water-tight by an impervious layer of clay. Filters of crushed stone, both fine and coarse, were placed on either side of the clay blanket to retain the material in place, on top of which was laid down a heavy layer of rip-rap.

The rock fill was placed by end dumping from trucks and sluicing with water, in the proportion of two of water to one of rock, with high pressure hydraulic monitors to achieve compaction and stability of the fill. The sloping filters on the dyke were placed in horizontal lifts by means of belly-dump trucks and compacted by controlling the movement of hauling and spreading equipment.

The placing and compaction of the impervious clay zone required close control both of the materials and the placing methods. Although some deposits of suitable silty clay were found within the reservoir that could be

Fig. 2.

used on this zone, the bulk of the clay was obtained by stripping a quarry site some six miles from the reservoir. The clay was dumped on the dyke from trucks and scrapers and then by means of bulldozers spread in position in comparatively thin layers ranging from 6 to 9 inches. When the surface of a previous placed layer became too dry or smooth, it was scarified and sprinkled to promote a better bond with the succeeding layer. The clay was compacted to the proper density by the use of sheep foot rollers.

Forming an integral part of the dyke is the pump storage generating plant (Fig. 3) located at the head of the canal. The width of the structure between the headworks and the tailrace piers is approximately 200 feet and the overall length of the powerhouse 570 feet, including an erection bay at one end for servicing the units. The six units in this plant are at 74 foot centres and are the largest in physical size in our system, with the exception of those at the St. Lawrence Project which are at 80 foot centres.

The pump generating station has no superstructure. This is the first station of this type to be built by Ontario Hydro. The roof deck of the powerhouse is located immediately above the top of the generators and access to the machines for major repairs is obtained through retractable hatches in the roof. The units are serviced by an outside gantry crane which traverses the full length of the powerhouse deck. This crane has a capacity of 175 tons and a housing of sufficient size to fully enclose one unit. This gantry was supplied by Dominion Bridge Company, and erected by Ontario Hydro.

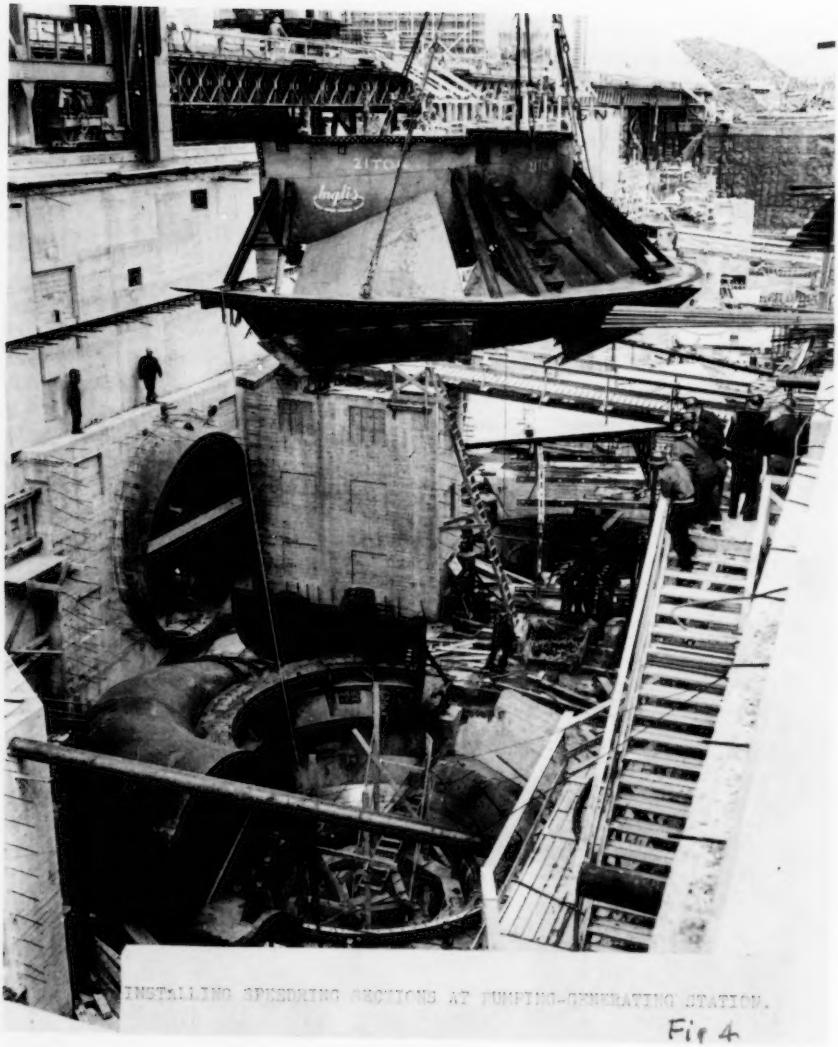
The steel penstocks which are approximately 60 feet long, vary in diameter from 19 feet at the scrollcase to 26 feet in the headworks transition. These penstocks were fabricated and installed by the Horton Steel Works Limited, Fort Erie, Ontario.

At the pump generating station are being installed six reversible feathering Francis type pump turbines, (Fig. 4) each directly connected to a motor generator. As pumps these units are rated at 55,000 h.p. and over the head range of 60 to 85 feet will discharge between 5,000 and 4,000 c.f.s. As turbines the units are rated at 47,500 h.p. under a net head of 85 feet. Over the full head range they will discharge between 5,600 and 4,100 c.f.s., or can be operated to give a continuous discharge of about 4,100 c.f.s. The speed of the machines is 92.3 r.p.m. With this equipment it is estimated that the reservoir can be filled in a period of about 7-1/2 hours.

The pump turbines are being supplied by John Inglis, Toronto, and installed by Ontario Hydro. (Fig. 5) The motor generators are being manufactured and installed by the Canadian Westinghouse Company. The pump turbine units have many differences in construction and in operation compared to the conventional units installed at the main plant. In the installation of the embedded parts the first unusual piece of equipment installed was the air admission nozzle at the heel of the draft tube. This is for the purpose of admitting outside air to the draft tube as hydraulic conditions may require. The next unusual part is the runner envelope; a large saucer shaped ring resting on top of the throat ring. This envelope has a 45° slope incorporating a spherical stainless steel clad surface which serves as clearance for the runner blades. This envelope must be set very carefully because of the small running clearance between it and the wheel. Following this comes the installation of the speed ring. This differs from the conventional machine in that the water passages are at 45° instead of horizontal. The stay vanes are also at 45° as are the six movable diffuser flaps which are extensions to the stationary

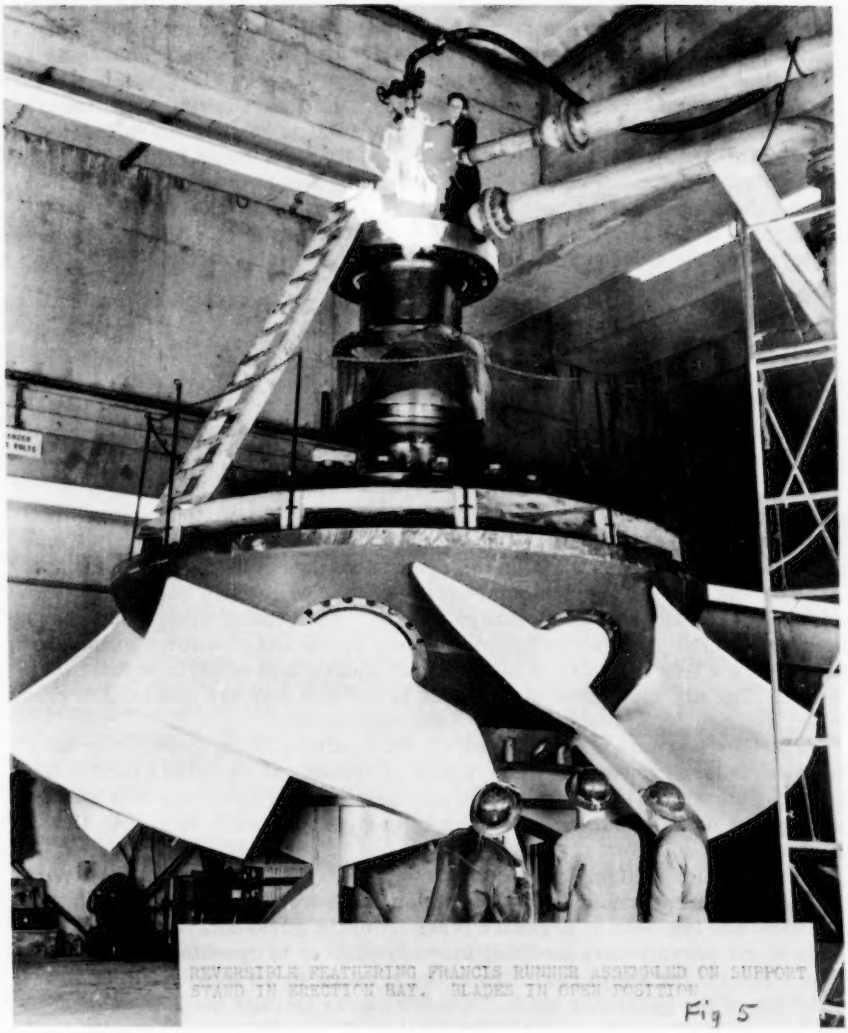
AERIAL VIEW OF PUMPING-GENERATING STATION CONSTRUCTION

Fig. 3



INSTALLING SPREADING NOZZLES AT PUMPING-GENERATING STATION.

Fig 4



REVERSIBLE FEATHERING FRANCIS RUNNER ASSEMBLED ON SUPPORT STAND IN ERECTION BAY. BLADES IN OPEN POSITION.

Fig 5

vanes. These flaps are used at heads below 60 feet when generating, but at high heads and when pumping are in line with the fixed vanes. There are no wicket gates in these machines, the water passages being controlled by the movement of the runner blades. Large steel columns are erected on top of the speed ring up to the generator bracket. The total weight of the generator and turbine moving parts is carried down through these columns to the stationary vanes and through the vanes to special footings in the concrete foundation below the speed ring.

The runner of these machines is an original design developed by the English Electric Company, of Rugby, England. It is known as a feathering type Francis wheel or a Deriaz runner after its designer, a Swiss engineer at the Rugby Works. Two models were built by the company and were successfully operated as both turbines and pumps. The first full scale machine built is the one being assembled here on the project. It was manufactured for the most part in Canada—the major exception being the blades which were machined in Switzerland.

Due to the runner's size and weight it was necessary to ship the component parts to the site and assemble them in the erection bay. The runner hub itself was the largest one piece casting ever made in Canada. It weighs 44 tons and when assembled holds 11 tons of lubricating oil. After the hub was placed on a support stand in the erection bay, the blades which operate at a 45^{o} angle were lined up with optical instruments on a launching ramp and drawn into position with a winch. Each blade weighs four tons and is bedded to close tolerances with the adjoining blades. The tips of the blades are stainless steel clad to reduce wear. Inside the hub is installed a large cylindrical servomotor to actuate the blades. The runner shaft weighs 27 tons and has an 8 foot 8 inch diameter flange at the connection to the runner hub.

After the assembly was completed the runner was tested under simulated operating conditions to check design figures. An actuator cabinet, a pressure tank, and a high pressure compressor were set up and connected to the runner to give a test pressure of 300 lbs. per square inch of oil to actuate the blades. The arc movement of the blade is 28^{o} and they are designed to close in ten seconds. The tests verified these conditions.

A further test was made to simulate the condition of the lubricating oil's centrifugal force when the runner would be rotating at its rated speed of 92.3 r.p.m. This was calculated to be 60 lbs. per square inch. This test was to assure that there would be no leakage through the oil seals when the machine was in operation.

The governor system consists of a single cabinet type actuator incorporating built-in pumps and sump tank with one pressure tank and two air tanks.

There are two sets of pressure pipes from the governor. One small set leads to six servomotors mounted in the headcover to operate the diffuser flaps. The larger set is routed from the actuator to a runner head located at the top of the generator shaft; from here down through the hollow generator and turbine shafts to the runner servomotor. The servomotor actuates the movable blades depending on the load on the machine and the head of water on the plant. In a closed position the blades act as wicket gates to close off the water passages.

When the machine is shut down, to reduce the water leakage past the blades, the clearance between the runner blades and the runner envelope is reduced by admitting water pressure from the scrollcase into a chamber in

the headcover; this introduces additional load on the generator thrust bearing which is so designed to depress slightly allowing the turbine blades to seal more closely to the runner envelope.

The motor generators differ from the generators at the main plant in that they use separate motor generator sets for excitation, specially designed transformer and motor windings for pump operation and full voltage starting as a motor. Ultimately, the pump generating units will be operated from the control room in the Sir Adam Beck Generating Station No. 2.

Construction of the pumphouse which started in the early part of 1955 is well advanced. Preparations are being made at this time to put into service the first of the six pump turbines this month; the remaining units will all be in operation by the early part of 1958.

Powerhouse Extension—Units 13 to 16

The remaining phase of the job still very actively under construction is the powerhouse extension. (Fig. 6) In January of last year the Commission authorized the start of four more units at this station to increase the plant capacity from 1,260,000 to 1,680,000 h.p. The first of these units is scheduled to go into operation in November of this year. This date coincides with the completion of the pump generating station and will enable more effective utilization of water from the reservoir.

To date concrete work has been completed on the draft tubes and the main walls of the substructure; erection of the penstocks is well advanced on the cliff; and the erection of the embedded parts for the first two turbines has been started. To unwater the substructure, sections of the steel crib cofferdam from the Grass Island Dam were used. Total concrete to be placed on the extension is 75,000 cubic yards and for the most part this is being placed using Ready-Mix trucks off two conveniently located Bailey bridges.

The four turbines for the extension are being supplied by the Dominion Engineering Company, two of the generators by the Canadian General Electric Company and two by the Canadian Westinghouse Company. The Horton Steel Company are erecting the penstocks. These penstocks are 19 feet in diameter and approximately 500 feet long. They vary in thickness from 5/8 of an inch at the top of the cliff to 1-1/2 inches at the connection to the scrollcase.

Upon completion of the extension this station will be 1,150 feet long with the main generator hall 63 feet wide and 50 feet high; housing 16 - 60 cycle generators each having a capacity of 80,500 Kilowatts coupled directly to 16 hydraulic turbines at 55 foot centres each having a capacity of 105,000 h.p., at a rated head of 292 feet.

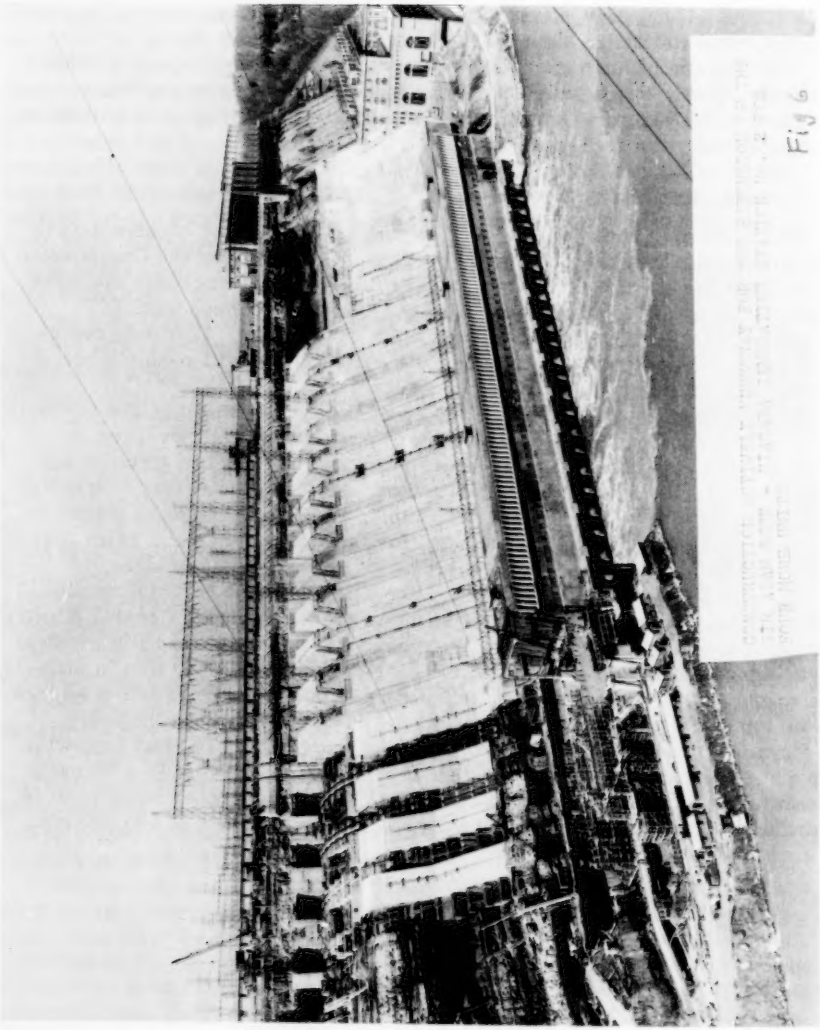


Fig 6

Paper 1420 October, 1957 PO 5

Journal of the
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Proceedings of the American Society of Civil Engineers

FIXED-WHEEL GATES FOR PENSTOCK INTAKES

Sylvan J. Skinner*
(Proc. Paper 1420)

SYNOPSIS

The general application of gates, valves and control mechanisms for hydro-electric installations are discussed in brief by articles in handbooks, text books and technical publications. Occasionally some specific feature or installation is covered in detail. In these publications the use and relative merits of the various types of gates are also discussed and, in general, agreement of type application is fairly consistent. However, there is a dearth of published information on the actual overall design of high-head flat structural steel fixed-wheel gates used for emergency closure of penstock or other conduit intakes. This paper covers the major design problems and the present design treatment by the Bureau of Reclamation for this type of gate.

INTRODUCTION

The designations "fixed-wheel gate" or "coaster gate," in general, refer to fabricated flat structural steel gates used for high head emergency applications; while radial gates, drum gates and roller gates are usually considered as surface regulating gates. The proportions and designs of fixed-wheel and coaster gates are determined primarily by their application for regulating flow in spillways or for emergency closure of the intake of penstocks, outlets, tunnels or in surge tanks. The intended function, operating facilities, and equipment to be provided for handling during installation and for later servicing also have an important part in the design. Their use is usually restricted to applications where emergency closure may be required to cut off flow of water through penstocks or conduits to protect other equipment in the system. In addition to the emergency feature their normal function is to close the conduit intake under balanced pressure conditions (no flow) so that the conduit

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may be unwatered for inspection or for inspection and servicing of turbines, valves and other equipment. They are usually raised (opened) under balanced pressures.

Principles of design and general construction of fixed-wheel and coaster gates are similar, the main variation being in the mountings. Coaster gates are mounted on roller trains instead of wheels, and can be operated under higher heads since the roller mountings offer much less frictional resistance to movement than bushed wheels. Coaster gates are more suitable for regulating purposes under high heads as they are less susceptible to vibration when operating for extended periods (at partial opening) with high velocity flow. More even distribution of load to the embedded gate frames and concrete structure is obtained with the coaster gate. Their main disadvantages are higher initial and maintenance costs, resulting from multiplicity of moving parts, and the accuracy of fit required for proper functioning. Therefore the more rugged fixed-wheel gates are used whenever possible.

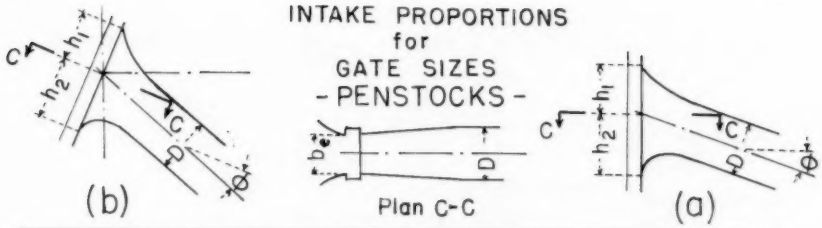
The gates are normally held in the open (stored) position, directly above the conduit intake, by hydraulic hoists located at the top of the dam or gate shaft. This position reduces the time required for emergency closure to a minimum. The hoists are connected to the gates by alloy steel or monel stems. The gates are lowered to the closed position under their own weight by releasing the supporting oil from the hoist cylinder, no downward force being exerted by the hoist. The design features of the "fixed-wheel gate" and its supporting frame, guides and anchorage for penstock intakes as currently treated by the Bureau of Reclamation are the subjects of the discussion that follows.

General Features Affecting Design

The principal considerations in the design of a fixed-wheel gate, as in any structure, are the magnitude of loading, the size of the gate, and the adequacy of its supports. The size of the gate (width and height) is determined by the size of the penstock or conduit and the shape and slope of the intake which the gate is to close (seal off from flow of water). The proportions of the trashrack and penstock intake structures are determined by flow characteristics necessary to obtai. the required "Q," flow in cubic feet per second, for operating the hydro-electric equipment at the power-plant. The design of the trashrack and intake structures is beyond the scope of this paper; however, the nominal size of the gate required to close the intake is determined by the proportions of the rectangular entrance of the transition to the circular penstock. The face of the rectangular transition entrance is usually coincident with the face of the dam and the downstream face of the gate slot. Fig. I (a) shows an intake with a vertical face; in Fig. I (b) the intake has a sloping face. The same nominal gate size will apply to an intake with either a vertical or a sloping face for the same size penstock, provided the angle θ, between the centerline of the penstock and the plane normal to the intake face, has the same magnitude.

The table of Fig. I lists intake proportions from which preliminary gate sizes may be obtained with sufficient accuracy for basing gate design stress computations and proportioning the various parts of the gate and its supporting frame. The table is based on the following:

INTAKE PROPORTIONS for GATE SIZES - PENSTOCKS -



PENSTOCK DIAMETER FEET D			ANGLE Ø DEGREES									
			0	5	10	15	20	25	30	35	40	45
	10	h_1	7.91	7.12	6.63	6.37	6.28	6.30	6.41	6.64	6.99	7.44
		h_2	7.91	8.01	8.17	8.40	8.70	9.09	9.57	10.19	10.98	11.97
		b_e	8.28	8.69	8.99	9.18	9.30	9.39	9.48	9.49	9.52	9.54
	11	h_1	8.70	7.83	7.30	7.01	6.92	6.93	7.05	7.32	7.69	8.18
		h_2	8.70	8.80	8.98	9.24	9.57	10.01	10.53	11.22	12.08	13.17
		b_e	9.10	9.62	9.88	10.10	10.22	10.33	10.42	10.43	10.46	10.48
	12	h_1	9.49	8.54	7.97	7.64	7.54	7.57	7.69	7.98	8.39	8.93
		h_2	9.49	9.61	9.80	10.08	10.44	10.92	11.48	12.24	13.18	14.37
		b_e	9.93	10.43	10.78	11.02	11.14	11.26	11.36	11.37	11.41	11.44
	13	h_1	10.28	9.25	8.63	8.28	8.17	8.19	8.33	8.64	9.09	9.68
		h_2	10.28	10.42	10.62	10.93	11.31	11.83	12.44	13.26	14.28	15.57
		b_e	10.76	11.28	11.66	11.93	12.06	12.20	12.30	12.33	12.36	12.40
	14	h_1	11.07	9.96	9.29	8.92	8.80	8.82	8.97	9.30	9.79	10.43
		h_2	11.07	11.22	11.44	11.76	12.17	12.74	13.39	14.27	15.38	16.76
		b_e	11.52	12.16	12.57	12.86	13.01	13.13	13.25	13.28	13.33	13.35
	15	h_1	11.86	10.67	9.95	9.55	9.43	9.45	9.61	9.97	10.49	11.16
		h_2	11.86	12.02	12.24	12.60	13.05	13.64	14.35	15.30	16.47	17.96
		b_e	12.42	13.04	13.50	13.76	13.94	14.10	14.20	14.23	14.26	14.31
	16	h_1	12.65	11.38	10.62	10.18	10.05	10.07	10.25	10.64	11.19	11.92
		h_2	12.65	12.82	13.06	13.45	13.93	14.55	15.30	16.32	17.57	19.16
		b_e	13.25	13.92	14.36	14.70	14.85	15.03	15.15	15.16	15.23	15.26
	17	h_1	13.44	12.10	11.28	10.83	10.68	10.72	10.89	11.30	11.89	12.66
		h_2	13.44	13.62	13.88	14.28	14.79	15.46	16.26	17.34	18.67	20.35
		b_e	14.07	14.78	15.26	15.62	15.80	15.97	16.10	16.13	16.16	16.20
	18	h_1	14.23	12.80	11.95	11.46	11.32	11.34	11.54	11.96	12.59	13.40
		h_2	14.23	14.42	14.69	15.13	15.66	16.37	17.22	18.35	19.76	21.55
		b_e	14.93	15.64	16.16	16.47	16.72	16.89	17.04	17.06	17.12	17.16
	19	h_1	15.03	13.52	12.51	12.10	11.95	11.96	12.17	12.63	13.29	14.15
		h_2	15.03	15.22	15.52	15.96	16.53	17.28	18.17	19.36	20.87	22.75
		b_e	15.74	16.52	17.13	17.46	17.65	17.86	18.00	18.04	18.09	18.13
	20	h_1	15.82	14.23	13.27	12.74	12.57	12.60	12.82	13.29	13.99	14.89
		h_2	15.82	16.02	16.33	16.80	17.40	18.19	19.13	20.38	21.95	23.95
		b_e	16.56	17.38	17.97	18.36	18.60	18.77	18.94	18.97	19.04	19.07

FIG.I For preliminary estimating only.

$$h_1 = \left[\sqrt{1.21 \tan^2 \theta + .0847} + \frac{1}{2 \cos \theta} - 1.10 \tan \theta\right] D$$

$$h_2 = \left[\frac{0.791}{\cos \theta} + .077 \tan \theta\right] D$$

$$\text{Opening Area} = \frac{\text{Penstock area}}{0.6 \cos \theta}$$

$$b_e = \frac{\text{Opening Area}}{h_1 + h_2}$$

D = nominal diameter of the penstock in feet.

h_1, h_2, b_e = transition face dimensions in feet. See Fig. I, a, b, & c.

The nominal size of the gate being b_e x (h_1 + h_2), in feet.

At an early stage in the proportioning of the intake structure and its emergency gate it is prudent to evaluate the operating requirements of the gate. This evaluation requires a determination of whether the gate can be closed by its own weight under free flow emergency conditions. Free flow emergency conditions are assumed to be equivalent to the flow from a burst penstock, if exposed, or to a run-away turbine with its wicket gates and protecting valve (if any) open and inoperative at the power plant. This evaluation is a compilation or summary of vertical forces existing during the closing and opening cycle of the gate. Insofar as operation of the gate is concerned the closing cycle is the more important as it determines whether closing can be accomplished by the gate weight alone. It also determines whether a normal fixed-wheel gate installation will be adequate or whether special considerations may be required, such as using retractable seals (which practically eliminate seal friction), installing a coaster gate, or providing a hoist with supported stems capable of exerting sufficient downward force to close the gate. Each of these special features add considerably to the cost. In such a summary the gate weight (submerged weight when downstream skinplate and seals are used) is the downward (positive) force P_p. The sum of the forces resisting downward movement is considered the negative force P_h. This negative force essentially comprises the wheel bearing friction F_b, the wheel rolling friction on the track F_r, and the seal friction on the embedded seal seats F_s. Of these, the wheel bearing friction F_b and the seal friction F_s are the most critical as they usually account for about 97% of the total negative force.

Experience shows that a minimum wheel diameter to pin diameter (wheel-pin) ratio of about 5 to 1 is normally required to obtain an acceptable amount of wheel bearing friction for gates operating under high heads. Seal friction is greatly reduced by bonding a comparatively thin brass nosing over the bulb of the seal, see Fig. II. Seals will be discussed in detail later. The wheel rolling friction F_r, being a very small percentage of the total negative force, is occasionally neglected in the preliminary summary of forces. Guide shoe friction is a negligible quantity since it can be no greater than a fraction of the guide shoe spring capacity. This spring capacity is seldom greater than two hundred pounds at maximum operating deflection.

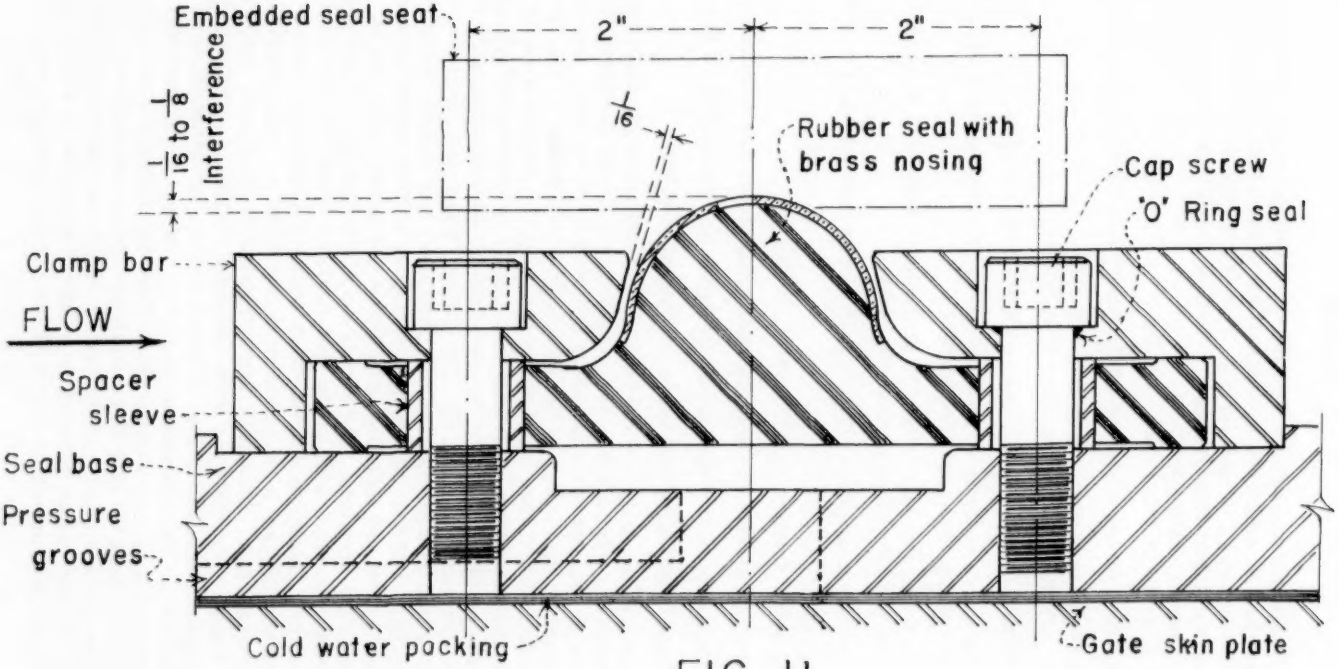


FIG. 11

TYPICAL SEAL ASSEMBLY

$$P_p - P_n = P_e$$

P_e = Excess downward force available for closing the gate by its own weight. An allowable minimum value of 25 percent of the total negative forces P_n is used for P_e. This is a safety factor to compensate for variations from assumed values of friction coefficients and unevaluated contingencies.
P_e (minimum allowable) = .25 P_n

P_p = Positive forces = Gate weight (submerged weight if applicable)

P_n = Summation of negative forces = $F_b + F_r + F_s$
F_b = Wheel bearing friction = $Wu_1/5$

W = Unbalanced hydrostatic load on closed gate (pounds).

u_1 = Coefficient of friction for wheel bearings, assumed as 0.10 for self-lubricating "Lubrite" type bushings.

5 = Wheel-pin ratio (assumed for preliminary summary of forces).
F_r = Wheel rolling friction = Wu_2/R_2

u_2 = Coefficient of rolling friction (assumed as 0.01 for steel on steel).

R_2 = Radius of wheels.
F_s = Seal friction = .217 $H_a u_3 1$ = .091 $H_a 1$

H_a = Average hydrostatic head, (at horizontal center line of gate).

u_3 = Coefficient of friction for rubber-brass seals (assumed as 0.42).

1 = Total length of rubber-brass seal on gate (inches).

For a vertical or nearly vertical face of structure there is usually no particular difficulty in obtaining a sufficiently adequate excess downward force F_e. A thorough analysis of the summary of forces is particularly important on installations where the face of the structure is on a comparatively flat slope. The fixed-wheel gate installations for the outlet and power tunnel intakes at the Palisades Dam, Palisades Idaho, are examples where special treatment was required to reduce the gate size and resulting hydraulic hoist capacity requirements, see Fig. III. The original plans proposed the use of the usual face type gate installation (see phantomed gate at face of intake in Fig. III) requiring 19.25' by 46' fixed-wheel gates with an estimated weight of 442,000 pounds each, and embedded gate frame and guides estimated at 200,000 pounds for each gate. The sum of the submerged gate weight component and the hydrostatic load on the gate produced wheel and seal friction resistance to gate closing movement which required a hydraulic hoist of extremely high capacity and a supported stem capable of exerting a heavy downward thrust on the gate. Design studies and hydraulic laboratory model tests (no formal report of the tests has been completed) indicated that the arrangement and proportions shown in Fig. III would be the most satisfactory all-around installation. The concrete beam "A" serves two major purposes. It permits retention of required intake proportions for desirable flow characteristics and relocates the gate slot a sufficient distance downstream from the face of the intake structure to materially reduce the gate size to 19.67' by 28.03'. This change in size reduced the weight of each gate to 280,000 pounds, a decrease of 162,000 pounds or nearly 36% of the estimated gate weight for

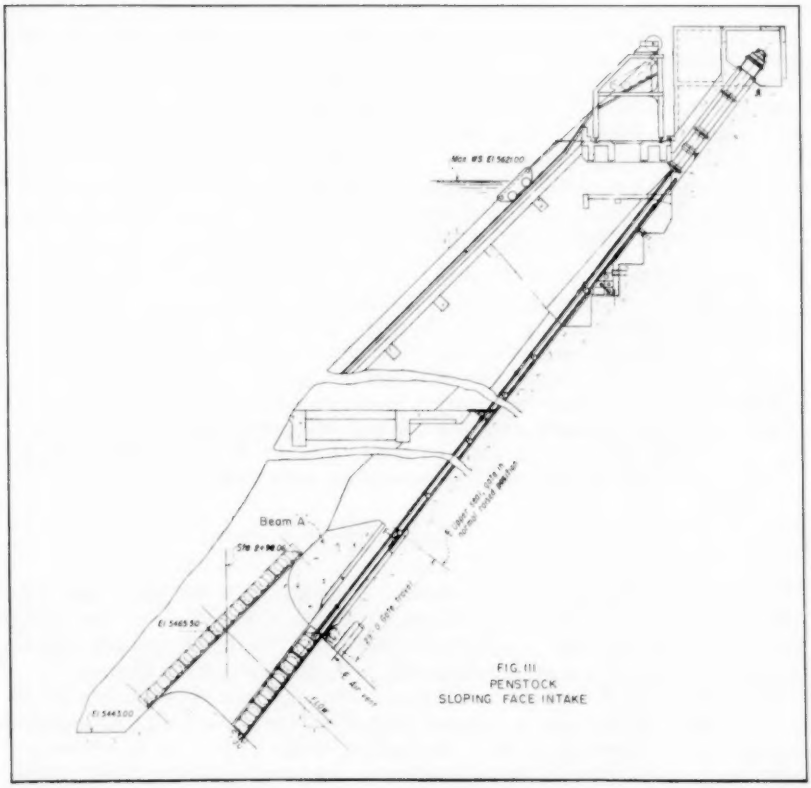


FIG. III
PENSTOCK
SLOPING FACE INTAKE

the original plan. The weight of the frame and guides was reduced to 102,500 pounds for each gate, nearly 50% decrease. Although the maximum head at the centerline of the gate was increased from 155.5 feet to 168 feet, due to the lower location in the intake, the reduced gate size decreased the total hydrostatic load on the gate by 33% with corresponding decreases in gate friction (F_n) and hydraulic hoist and stem thrust effort needed to close the gate.

An estimated gate weight for the gate size and operating head contemplated can be obtained from Fig. IV. This chart is based on calculated weights of various sizes of fixed-wheel and coaster gates which have been designed, built, and installed for operation under varying heads at conduit intakes. The weight obtained from the chart will be reasonably accurate provided the design does not deviate appreciably from the general type of construction of the gates from which data were obtained for use in developing the chart. The gate design discussions included herein apply to this type of construction.

The following nomenclature applied to Fig. IV.

H = Head to bottom seal, feet
L = Width of opening, feet. (b_e, Fig. I)
T = Height of opening, feet. ($h_1 + h_2$, Fig. I)
X = Height of gate frame, feet. (Assume 2.3 T).
W/T = Weight of gate per foot of height T.
W/X = Weight of frame per foot of height X.
Note - Weight of guides are estimated at 100 pounds per foot of height, [assumed to be X + (2 to 5 feet)]. Add to frame weight to obtain total estimated weight of embedded metalwork.

Downpull

One of the important factors involved in the design of the gate, and especially in the design of its operating equipment, is the downpull force which might obtain during the emergency closing or opening. During emergency operation the gate is subjected to large unbalanced pressures resulting from an increase in velocity of flow under the gate. The upstream face and top of the gate are subjected to nearly constant pressures while there is a decrease in pressure on the downstream face and bottom of the gate. The decrease in pressure on the bottom of the gate produces an unbalance of vertical pressures resulting in a downward force on the gate generally referred to as downpull. The pressure variations on the bottom of the gate are functions of several factors: the shape of the intake; the velocity of flow under the gate; the angle at which the flow approaches the opening under the gate; the proportions of the gate bottom; the hydraulic conditions downstream of the gate; the amount of gate opening; the relative positions of the top and bottom seals; and the proportions of the recess in the face of the structure just above the conduit entrance. These factors all influence the flow pattern and the velocity distribution under the gate.

Since the pressure reduction at any point is controlled by the velocity distribution and is equal to the velocity head at that point, the downpull force is dependent on the summation of these velocity heads. There are no known available data which give sufficient information to determine accurate estimates of downpull values for various types of gates at any gate opening. Laboratory hydraulic model tests and studies, based on proportions of the

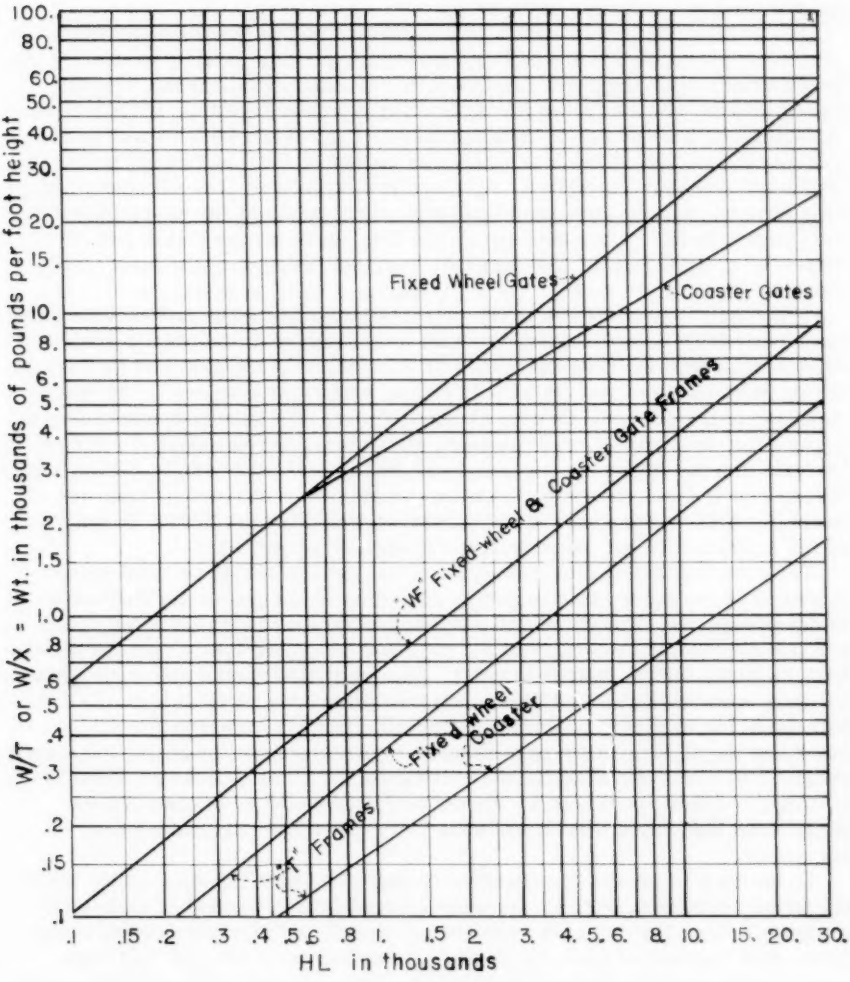


FIG. IV

WEIGHT OF COASTER & FIXED-WHEEL GATES

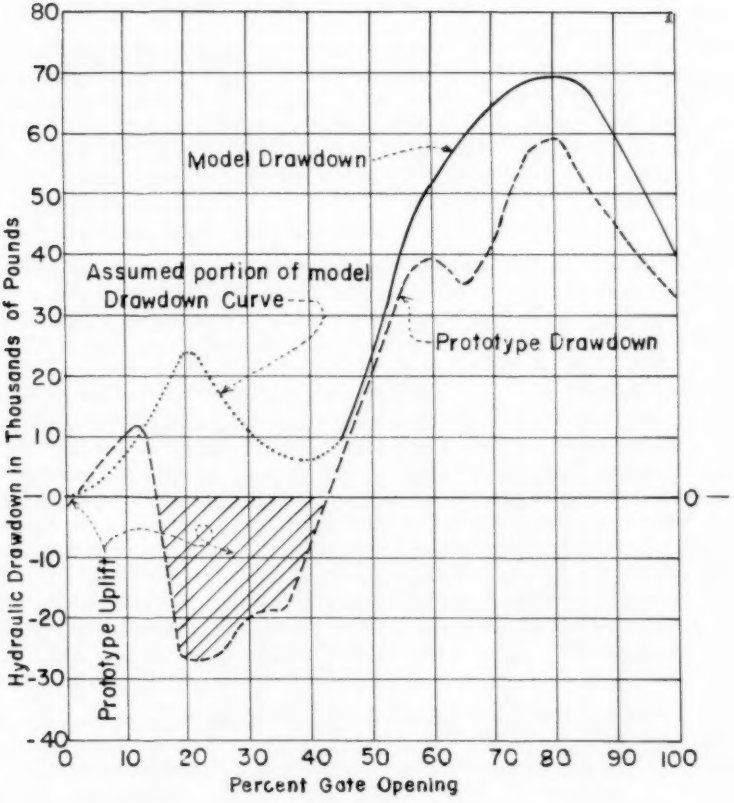
proposed gate design, are at present the most satisfactory methods of determining a reasonably accurate estimate of the downpull force that can be expected for any particular installation.

Model tests so far conducted have been confined to those installations where necessary to solve a difficult downpull problem particular to a specific installation. Such tests are invaluable for similar installations, yet they serve at best only as guides for estimating downpull for other installations. Assumptions are necessary to adapt the tests generally with no means of proving the validity of such assumptions unless or until additional model of prototype tests are conducted. Although prototype tests are not usually made, they have been conducted occasionally to verify some of the special laboratory model tests. In one such prototype test, uplift forces on the gate were observed at certain gate openings that were not covered in the model tests, see Fig. V and Reference 1.

The physical proportions of the gate are generally large and require comparatively little additional construction to accommodate increased loads and stresses induced by downpull forces on the bottom of the gate and on the gate lifting stem connections. In view of the uncertainties involved in the determination of the downpull forces, as noted above, it is assumed reasonable to apply a full hydrostatic head pressure differential and design the bottom of the gate and the lifting stem connections on the gate accordingly. Since such forces result from emergency closures which occur infrequently and are of short duration, an increase in the allowable unit stresses to 75% of the yield of the material for the parts involved is usually permitted.

The effect of downpull on the operating equipment would be most severe if it should be necessary to completely open (raise) the gate under full unbalanced pressures. The maximum downpull force would then be added to all friction forces and the weight of the gate and hoist stems to obtain the necessary capacity for the hydraulic hoist. This condition should not occur since the gate is normally raised under balanced pressures. Provisions are usually made for obtaining balanced pressures, after an unwatering of the penstock, by installation of by-pass lines and control valves for filling the empty penstock to reservoir head before raising the gate. An occasional installation may require the gate to be opened a slight amount to accomplish the backfilling, but only where installation of by-pass lines and valves are unfeasible.

An air vent is provided, immediately downstream of the gate, in the top of the intake transition, Fig. VI. It supplies air which is needed to reduce the tendency toward formation of negative pressures and vibration during emergency gate closure or when the conduit is drained after the gate has been closed under normal balanced pressures. It permits the release of air while the conduit is being filled with the gate closed or nearly closed. The air vent also relieves the tendency toward formation of partial vacuums in the conduit during normal flow conditions. Since the air vent relieves the tendency toward producing negative pressures on the downstream face of the gate, the maximum hydrostatic head on the gate is increased by only 15 feet to obtain a design head which allows a safety factor for a possible partial vacuum on the moving gate. The minimum size of the air vent should be sufficient to admit air at the rate the turbine is discharging water during emergency closing of the gate (assuming head water at the centerline of the transition and 1/2 atmosphere pressure on the downstream side of the gate).



SHASTA DAM
Outlet Coaster Gate

FIG. V

Model - Prototype Comparison of Hydraulic Drawdown
(See Ref. 1)

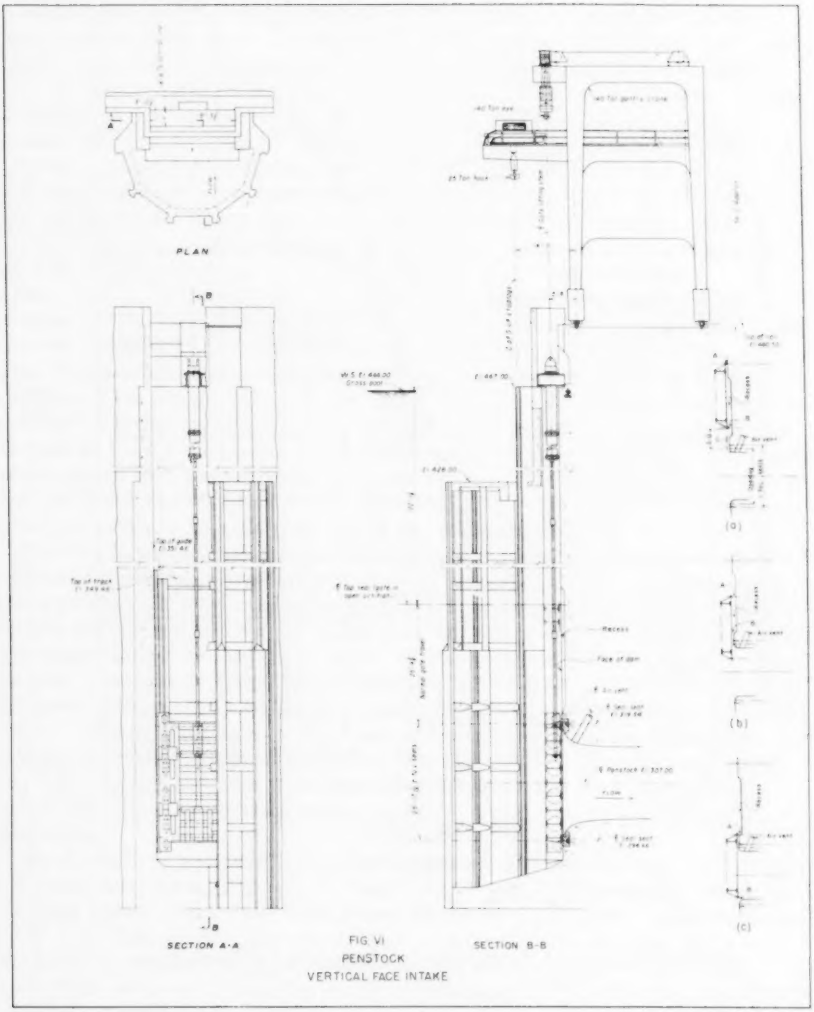


FIG. VI
PENSTOCK
VERTICAL FACE INTAKE

A recess is built in the face of the concrete structure just above the intake and behind the gate in its normal raised position, see Fig. VI. This recess serves the following several purposes and should be properly proportioned to serve each to the best advantage.

1. With the gate in its normal raised (open) position, the recess serves to dampen gate vibration. This is accomplished by establishing the control of flow, between the gate and the concrete structure, at the orifice formed by the top seal on the gate and the concrete structure; i.e. orifice "A" must be smaller than "B," Fig. VI a. Under flow conditions this produces a reduced pressure on the downstream face of the gate, between the gate and the recess, thus loading the gate and preventing vibration or flutter of the gate resulting from effects of the intake flow pattern.

2. The seal assembly on the gate projects a minimum of 2-3/4 inches beyond the face of the skin plate and without a recess the top seal assembly will be subjected to unbalanced pressures when the gate is partly closed (full reservoir pressure on the top side of the assembly and lower pressure on the under side). This unbalance of pressures on the top seal assembly contributes materially to the downpull force on the gate. The maximum downpull force generally occurs between gate openings of 20 to 80 percent. Tests (Ref. 2) have shown that a recess with a depth several times greater than the seal assembly projection is effective in reducing downpull by balancing the pressures on the top seal assembly. The effectiveness of the recess in balancing pressures is directly proportional to its depth. A recess with a depth equal to 3 or 4 times the projection of the seal assembly is usually built in the concrete structure and proportioned to be effective approximately through the 20 to 80 percent gate opening range, except in cases where results of special model tests indicate other requirements. The recess is effective for this purpose through the gate travel where orifice "A" is greater than orifice "B," Fig. VI b, as noted above.

3. As a closing gate approaches the closed position the spring point of flow jumps from the lip on the bottom of the gate to the bottom seal assembly. This shift of the spring point produces an unbalance of pressures on the bottom seal assembly similar to those which would occur on the top seal assembly with the gate open and no recess. Since the hydrostatic head is effective on the lower side of the bottom seal assembly and a reduced pressure (near atmospheric as air is supplied thru air vent) exists on the upper side, an uplift force on the projection of the bottom seal assembly results. The uplift can be sufficient to offset the otherwise net excess downward force "P_e" and stall the gate before it is completely closed. To counteract this uplift force the effect of the recess in balancing the pressures on the top seal assembly is again taken advantage of. Since the pressure balancing effect of the recess on the top seal is directly proportional to its depth, the depth can be reduced near the end of the closing cycle so as to produce a downward force on the top seal assembly to counteract the uplift on the bottom seal assembly, orifice "A" is less than orifice "B," Fig. VI c.

The above functions of the recess are the major factors in determining the normal raised (open) position of the gate. For obtaining the shortest distance of travel for closing and the resulting minimum size for the hydraulic hoist the preferred position for the raised (open) gate would be determined by locating the tangent point (spring point of flow, i.e. either the upstream face of the

TABLE 1
Allowable Unit Stresses

	Pounds per sq. in.	
	Members embedded in concrete	Gate members
Axial tension (net section)	18,000	15,000
*Axial tension - power driven rivets	- ---	5,000
Axial tension - anchor bolts.	15,000	- ---
Tension - Extreme fibers - (Bending).	18,000	15,000
Compression - Extreme fibers - (Bending)		
l/b less than 15.	16,500	14,000
l = length of unsupported flange between connections - inches		
b = flange width - inches		
Combined stress	24,000	20,000
Shear - Gross area webs	11,000	9,500
Shear - power driven rivets - fitted bolts - Pins.	13,500	11,500
Shear - Ribbed bolts (70,000 psi.min.Ult. Tens.).	18,000	15,000
Shear - Turned bolts.	11,000	9,500
Shear - Tangential in Wheels.	- ---	90,000
Bearing - Pins.	24,000	20,000
Bearing - Power driven rivets, ribbed bolts tight fit dowels & bolts, milled stiffs	27,000	23,000
Bearing - Turned bolts.	20,000	17,000
Bearing - "Lubrite" type bushings - 10 fpm.	- ---	4,000
- Static.	- ---	7,000
Bearing - Concrete in blockouts under track bases	3/8 of Ult. strength	- ---

*Allowable tensile stress not over 5,000 psi or

$S_t = (4630\ T_r/V_r - 8000\ d_r + 11,500)\ 0.8$**, whichever is the lesser.

S_t = Permissible tensile stress on rivet in psi of rivet before driving.
T_r = Total tension on rivet, - pounds
V_r = Total shear on rivet, - pounds
d_r = Diameter of rivet before driving, - inches

**Derived from Reference 3.

gate or the low edge of the skin plate) on the gate to coincide with the vertex of an ellipse which is identical to the ellipse forming the top of the penstock intake. The position of the ellipse at the spring point of flow on the gate being determined by moving the vertex of the penstock ellipse along a line parallel to the centerline of the penstock until it is tangent to a vertical line drawn through the spring point of flow on the gate. The major axes of the ellipses remain horizontal at all locations, see Fig. VI. If these conditions cannot be met then the gate should be raised to a location approximately three feet higher than the preferred position described above so that it will not materially affect the flow pattern through the penstock intake. In all installations where a recess is required for dampening of gate vibration it will be necessary to raise the gate to the higher open position, because of the requirement that orifice "A" must be less than orifice "B" Fig. VI a.

Unit Stresses

Allowable working stresses for the gate leaf are proportioned to compensate for corrosion. Previously, stresses were computed for sections from which 1/32 inch had been deducted for exposed surfaces, a procedure requiring a special moment of inertia computation of standard rolled structural sections for the corrosion allowance. From comparisons of stresses on previous designs, before and after the 1/32 inch deductions, the preceding tabulated allowable unit stresses (which include corrosion allowances were applicable) were adopted.

Gate Leaf

A fixed-wheel gate leaf is essentially a bulkhead gate mounted on wheels, and consists of a skinplate supported by horizontal beams connected to vertical end beams usually referred to as vertical girders. The wheels are mounted between two vertical girders at each end of the gate. Eccentric pins are used for mounting the wheels, the eccentric being arranged to provide adjustment of wheel alinement so that the tread contact surface of all wheels on the gate can be alined to a common plane for bearing on the embedded track. A typical slot section showing the relative positions of these parts is shown in Fig. VII.

Three basic arrangements of the horizontal beams, vertical girders and wheels are used. These are shown in Fig. VIII (a), (b), and (c). In (a), all of the horizontal beams extend through to the outside vertical girders, thus placing a wheel between each pair of horizontal beams. This arrangement is suitable for low operating heads where the wheel loads are comparatively light and no particular difficulty is encountered in obtaining a sufficiently large wheel-pin ratio to keep the wheel bearing friction F_b within usable limits. The use of load distributing beams (or diaphragms) in (b) and (c) provide additional space for the use of larger wheels which are needed for higher heads. They permit the retention of close horizontal beam spacing which reduces the thickness of the skin plate. Their use also obviates the need for development of complicated connections at the eccentric pin nuts, pin bearing plates and end connections for the horizontal beams, a condition that would exist if the horizontal beams extended to the inside vertical girder. The large wheel chamber provided in arrangement (c) adds flexibility to the

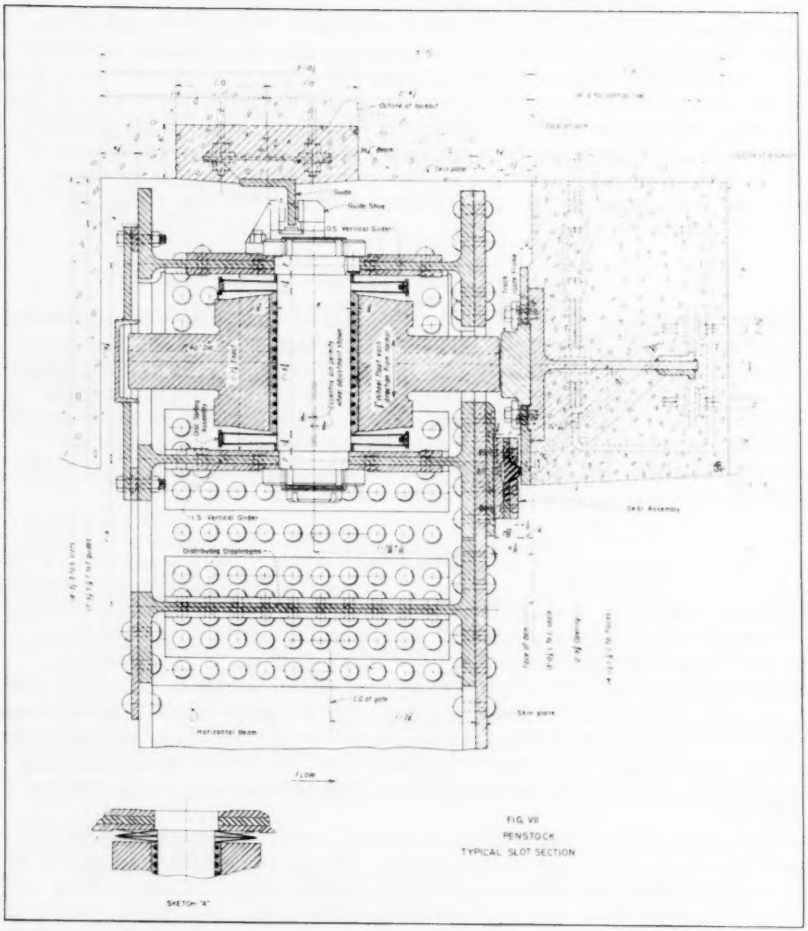
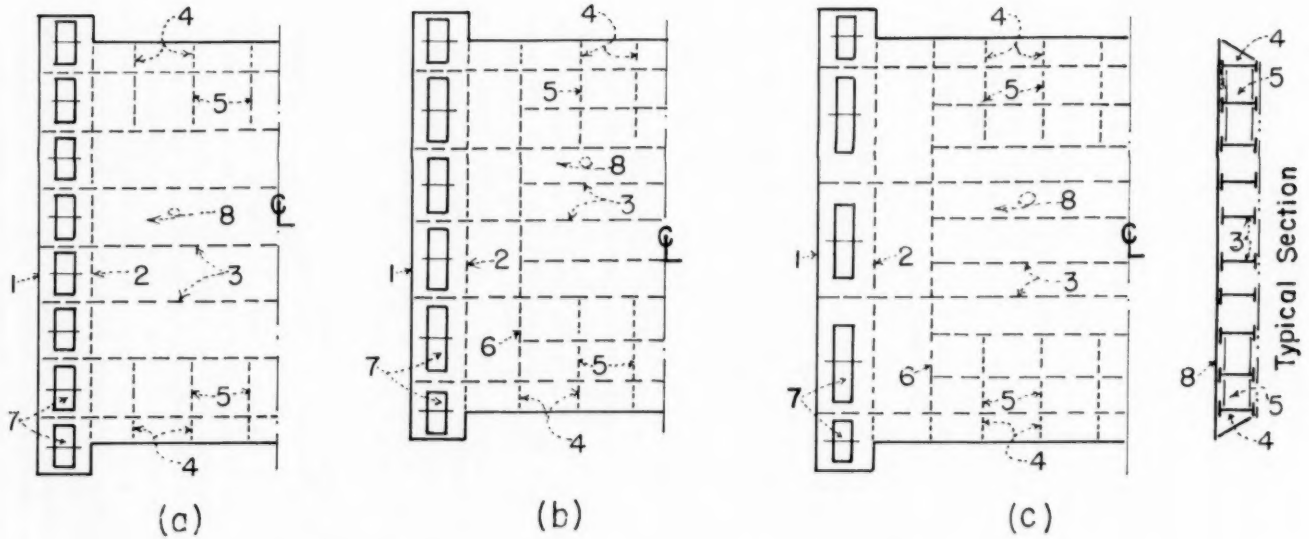


FIG. VII
PENSTOCK
TYPICAL SLOT SECTION



1 - O.S. Vertical Girder
2 - I. S. Vertical Girder
3 - Horizontal Beams
4 - Cantilever Brackets
5 - Diaphragms
6 - Distributing Beams
7 - Wheels
8 - Skinplate

PENSTOCK FIXED-WHEEL GATES

FIG. VIII

location of the wheel. In some cases this is a distinct advantage in that it provides a simple means of redistributing the wheel loads by utilizing the effect of wheel spacing on the distribution of the vertical girder loads to the wheels. The arrangements can be used in combinations to suit the size of the gate and the design head.

The roller trains on coaster gates are located over the center-line of a single vertical girder at each side of the gate. The gate leaf load is evenly distributed throughout the length of the roller trains and onto the tracks embedded in the concrete structure.

The design of the major parts of the gate leaf is a procedure of trial and error, previous designs being invaluable in choosing the preliminary trial sections. The span length of all through-horizontal beams is assumed to be the center to center of wheels. The spans are loaded between the center lines of the vertical seals.

The skinplate moments, loads and bending stresses are obtained by treating a vertical unit width strip of the skinplate as a continuous beam with reinforcing haunches and modified end cantilevers (Fig. VIII). The flanges of the horizontal WF beams are the haunches with an effective width equal to the distance between the centers of the extreme rivets in the flange plus one rivet diameter. The parts of the skinplate which extend above the top beam and below the bottom beam are cantilevers, supported laterally at frequent intervals by brackets which are connected to the horizontal beams. These cantilever portions of the skinplate are analyzed as a series of rectangular plates, each simply supported on two opposite edges at a bracket, the third edge fixed (built in) at the horizontal beam, and the fourth edge free, (Ref. 4). Initial end moments for the continuous skinplate haunched beam and the resisting moments at the center of the fixed edges of the top and bottom rectangular plate cantilever are calculated. These moments are used to obtain the actual moments at the supports by the method of moment distribution (Reference 5, Chapters 3 and 5). By combining these actual moments with the simple moments of the spans the moment at any point on the continuous skinplate beam may be obtained. Relatively close spacing of the brackets under the cantilever portions of the skinplate eliminates the high fixed-end moment of a free cantilever and permits a greater extension of the skinplate beyond the horizontal beam. This flexibility in the length of the skinplate extension and initial resisting moment can be used to advantage in the moment distribution to obtain resulting horizontal beam loads that are acceptable.

The shear diagram of the continuous skinplate beam is a composit of the simple span shears at the supports and the shear due to continuity in each span. The shear due to continuity is the algebraic sum of the actual moments at the supports of the span divided by the span length. It is added to the simple shear at the support with the greater numerical moment and is subtracted from the simple shear at the other support. The resultant total shear at each support is the distributed load, per lineal inch, on the supporting longitudinal beam. Fig. IX shows a typical moment distribution with moment and shear diagrams. Fig. IX (a) shows the comparable moment distribution, with moment and shear diagrams, for a free cantilever skinplate. The stresses in the skinplate and horizontal beams are calculated from the moments and shears thus obtained. The skinplate is attached to the flanges of the horizontal beams and adds to the moment of inertia of each composit beam. It is therefore subjected to biaxial stresses; those resulting from

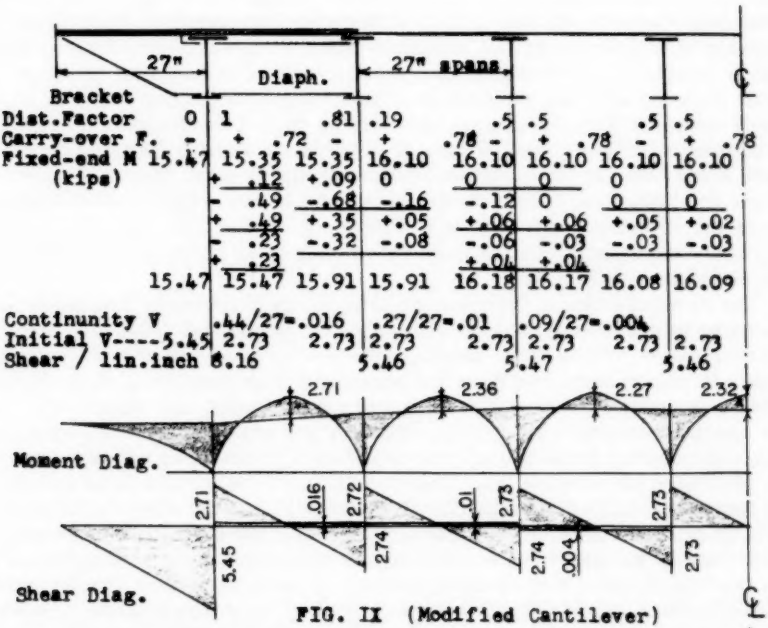


FIG. IX (Modified Cantilever)

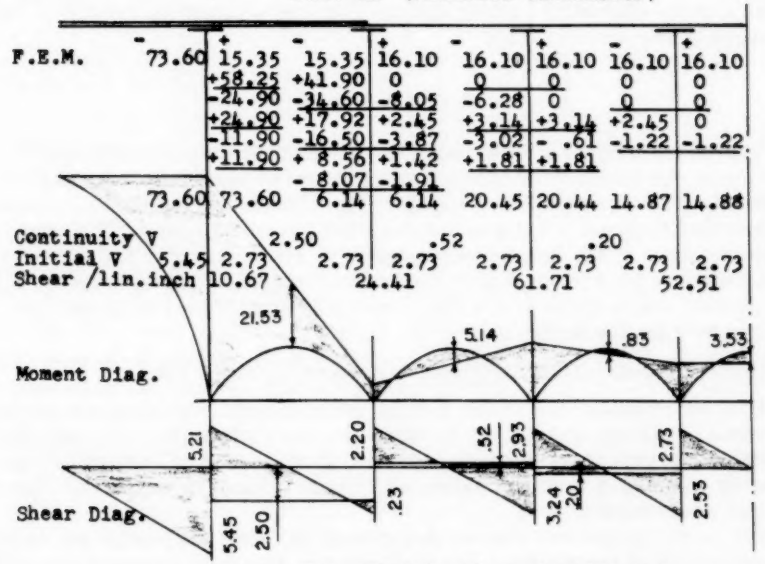


FIG. IX (a) (Free Cantilever)

horizontal beam bending and those from the skinplate bending. The maximum combined stress in the skinplate is determined according to Westergaard's criteria of failure for ductile materials subjected to two-dimensional stress and is limited to 20,000 pounds per square inch. Comparisons have been made between different methods of determining what portion of the skinplate might reasonably be considered effective as part of the horizontal beam area in determining the moment of inertia of the beam section. An effective skinplate width equal to the horizontal beam spacing is used, except in cases where the horizontal beam spacing is over 30 inches. For these exceptional cases the effective width may be limited to .11 of the horizontal beam span; or, where the span is relatively short a factor of 30 times the thickness of the skinplate may be used.

The vertical girders, distributing beams, and diaphragms are made from the same size rolled structural section as the horizontal beams to simplify the fabrication. Since these members have very short spans their principal stress is shear; and, even though a considerable percentage of their flanges may be removed for wheel clearances or access holes, the bending stresses are usually very low. The vertical girders are analyzed as continuous beams with concentrated loads at the horizontal beam connections and supports at the wheel and pin assemblies. The moments and shears are determined by the method of moment distribution (Reference 5) and the wheel loads taken directly from the results. As stated previously, the magnitude of the wheel loads may be redistributed by altering the wheel spacing if sufficient room has been provided at the vertical girders. In some instances where considerable difference in wheel loads had been obtained from the original computations a more equivalent loading distribution was obtained by relatively small changes in the wheel spacing.

Wheel Assembly

The wheels are made of wrought steel, (A.S.T.M. Specifications A 57) with minimum rim hardness of 255 B.H.N. The stress in the wheel tread is the controlling factor in the wheel design. The diameter of the wheel is usually determined by practical limits and the available wheel space. The required projected area of the wheel tread (product of wheel diameter and tread width) is determined by a formula developed from tests on rollers (Reference 6), which has been found to give a conservative critical stress in pounds per square inch for the design of wheels:

Critical Stress = (Brinell hardness number) X 24.5 - 2200

A factor of safety of 2 on the critical stress is assumed adequate for an overload condition, where any one wheel on a side of the gate may not bear on the track for a short distance of travel, thus causing an overload on the adjacent wheels. A factor of safety of 3 on the critical stress is used for the normal wheel load.

The larger projected area, as determined by these two conditions, is used for the design of the wheels. The stresses are analyzed in accordance with the method derived in Reference 7 for crossed cylinders, from which the chart of Fig. X was taken. The following nomenclature applies to this chart:

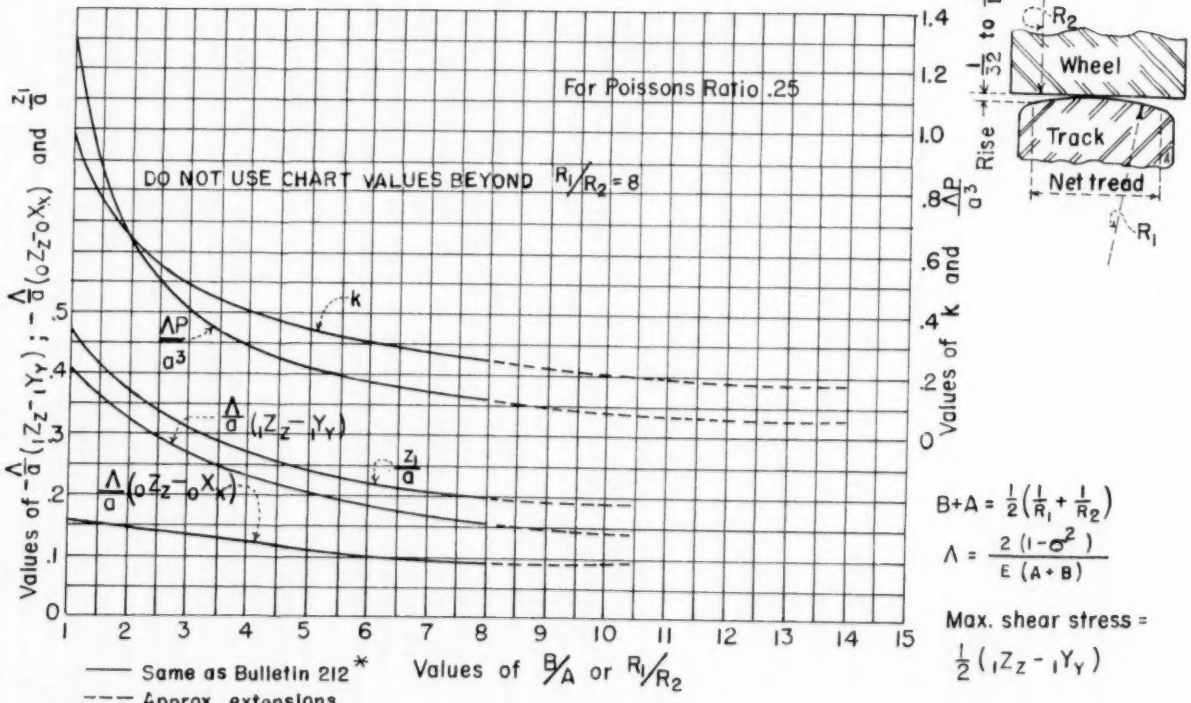

For Poissons Ratio .25
DO NOT USE CHART VALUES BEYOND $R_1/R_2 = 8$
Values of $-\frac{\Delta}{a}({}_1Z_Z - {}_1Y_Y)$; $-\frac{\Delta}{a}({}_0Z_Z - {}_0X_X)$ and $\frac{z_1}{a}$
Values of $\frac{\Delta P}{a^3}$ and k
$\frac{\Delta P}{a^3}$
k
$\frac{\Delta}{a}({}_1Z_Z - {}_1Y_Y)$
$\frac{z_1}{a}$
$\frac{\Delta}{a}({}_0Z_Z - {}_0X_X)$
Values of B/A or R_1/R_2
Same as Bulletin 212*
Approx. extensions
*University of Illinois Engineering Experimental Station
R_2
Wheel
$\frac{1}{32}$ to $\frac{1}{16}$
Rise
Track
Net tread
R_1
$B+A = \frac{1}{2}\left(\frac{1}{R_1} + \frac{1}{R_2}\right)$
$\Lambda = \frac{2(1-\sigma^2)}{E(A+B)}$
Max. shear stress = $\frac{1}{2}({}_1Z_Z - {}_1Y_Y)$

FIG. X

P_w = Wheel load, pounds

R_1 = Track radius, inches

R_2 = Wheel radius, inches

A = Mean of reciprocals of radii in X-direction = $1/2R_1$

B = Mean of reciprocals of radii in Y-direction = $1/2\ R_2$

E = Modulus of elasticity = 30×10^6 pounds per square inch

σ = Poisson's ratio = 1/4

$\Lambda = \frac{2\ (1\ -\sigma^2)}{E\ (A + B)}$ = Evaluation of elastic properties and shape properties

a = semimajor axis of the ellipse of contact

b = semiminor axis of the ellipse of contact

z_1 = Depth to point of maximum stress difference or point at which maximum shearing stress occurs.

The maximum difference of stress components is $({}_1Z_z - {}_1Y_y)$ and the maximum shearing stress is $1/2\ ({}_1Z_z - {}_1Y_y)$. Data on the maximum shearing stress allowable are somewhat indefinite as stated in Reference 7; however, it is generally assumed that about 90,000 psi is safe. A fixed-wheel gate is operated very infrequently under emergency conditions and then at slow speeds; the wheels will seldom be used under full load, and the condition of wheel overload is remote. Therefore the 90,000 psi is considered conservative. The required rim hardness of 255 B.H.N. is usually specified to penetrate at least 1-1/2 to 2 times the depth Z_1.

The eccentric pins for the wheels are either made of solid precipitation hardening stainless steel (17% Cr., 7% Ni., Type 322, USS Stainless W or equal), or high strength alloy carbon steel with 1/32 inch thick electrolytically nickel plated bearing surfaces. During the years when the use of nickel was restricted pins were made of heat treated solid stainless steel (12% Cr.) type 410 or 416 and the alternate alloy carbon steel pins were chrome plated .005 to .010 inch thick. However, the chrome-nickel stainless pins or nickel plated pins are used if obtainable because they are less susceptible to corrosion and electrochemical action when used with "Lubrite" type bushings in constantly submerged installations.

These pin materials are specified to have minimum physical properties in the ranges of 110,000 to 120,000 psi Ultimate Tensile Strength and 75,000 to 90,000 psi Yield Point in Tension. An allowable bending fiber stress of 33,000 psi is used for the pins, which are designed as partially uniformly loaded simple beams. The maximum deflection of the fully loaded pin must be less than the coarse running fit clearance between the bushing and the pin. The bushings are made of high strength manganese bronze with self-lubricating graphite inserts (Lubrite type). The graphite inserts are pressed into drilled recesses in the bushing wall. The recesses do not extend through the wall, thus providing a solid backing for the lubricant and a stronger bushing. They are designed with an allowable average bearing stress of 3500 to 4000 psi on the projected area and are specified to have a minimum static load capacity of 7000 psi. They are made as two-piece split bushings and are press fitted into the wheels.

Disc springs are also included as a part of the wheel assembly. Their

function is to center the wheel on its pin when it is unloaded and free to move laterally. A wheel that is centered with adequate clearance on each side of the hub, just prior to closing the gate under emergency conditions, is free to float sidewise, a movement that may result from any slight misalinement in pin bores or in the track. Once a wheel has taken load it is impossible to change its lateral course because of the high friction between the tread and track. The wheel float avoids a buildup of friction between the wheel hub and the vertical girder (which would tend to stall gate movement) by preventing contact of these parts while the gate is moving under load. Initially flat uniform section disc springs arranged as shown in Fig. VII are provided for each wheel. The springs are made of 3/4 hard 18% Cr. - 8% Ni. or precipitation hardening 17% Cr. - 7% Ni. stainless steel sheet or strip. They are designed for installation with an initial preload deflection sufficient to produce a minimum load equalto the force required to slide 1-1/2 times the combined wheel and bushing weight on the pin, assuming a sliding friction coefficient of .10 between the bushing and pin. Stress in the springs, at maximum deflection equal to half the assumed wheel float, is limited to 125% of the Yield Point in Tension of the spring material. The procedure of spring design otherwise follows that developed by Almen and Laszlo, Reference 8.

Initially flat springs are used instead of the predished or initially coned type as they are simpler to manufacture and, as a group, are more consistent in their load capacities. This is probably due to elimination of the preforming operation required in the coned type. Also, the assembly of flat springs maintains a compact closed unit when released of load, making it less susceptible to accumulation of sediment. The initially coned type produces a higher initial load under very little deflection, which is a desirable characteristic. However, some installations of this type have encountered difficulty with the cone snapping-through under load and the spring rendered useless. A combination, using a flat disc as a diaphragm between the initially coned springs would keep them effective, even though a snap-through should occur, provided all outer edges at the inside and outside diameters of the paired springs are supported when the cone is pointed in either direction, see Sketch "A" Fig. VII. This requires designing both types of springs with no apparent advantage over the initially flat spring assembly.

Guides are installed in the concrete structure and engaging spring-backed guide shoes are attached to the four corners of the gate. The guides and shoes serve a similar function to the gate as a whole as the disc springs do to the wheels; they center the unloaded gate in the gate slot above the intake opening and allow lateral movement of the gate during emergency closure. They also center the gate in the lowered position after the penstock is backfilled and the pressures on the gate balanced prior to raising it to the normal open position. The metal guides extend from the sill at the intake to about five feet above the gate when in the raised position. Above this point the gate is guided by shoes rubbing on the concrete surfaces of the gate slots as no close guiding tolerances are required in this area.

Seal Assembly

For recent high head gate installations the double-stem seal assembly shown in Fig. II has replaced all types of assemblies previously using single-stem "J" or music-note seals. The solid bulb music-note seal is now used

almost exclusively for bulkhead gate installations. The double-stem seal is attached to the seal base with clamp bars over the stems. All parts are proportioned to provide adequate clearances around the seal to permit flexing action of the stems and extension of the seal bulb to contact the embedded seal seats. Reservoir pressure is admitted to the chamber behind the seal through slots or grooves milled in the seal base. Since half of the seal bulb is under balanced pressure (reservoir head) the seal load on the seal seat is one-half of the total reservoir pressure on the unbalanced side of the bulb, assuming the clamp bar as taking its portion of the total unbalanced load from the seal acting as a beam.

The brass nosing is bonded on the bulb and serves to reduce friction during an emergency closure of the gate. It also prevents distortion and displacement of the top and bottom seals when they make contact with the horizontal seats while extended under such closures. To guard against leakage as effectively as possible, rubber "O"-ring seals are used for all fastenings on the unbalanced pressure side of the seal, and a cold water sheet packing gasket is provided between the seal base and the gate leaf. Field reports from project installations indicate that this type of seal and assembly is performing very efficiently.

Some emergency gate installations require the seals to be unloaded (sometimes referred to as retracted seals) until the gate is fully closed, to reduce the seal-to-seal seat friction to a minimum. This is accomplished by using a hydraulically actuated seal controlled by a small regulating valve. This is a three-way type of valve with a port leading to the seal chamber, another to the low pressure side of the gate, and the third to the high pressure or reservoir side. A short overtravel of the hoist and gate stem actuates the valve. When the gate is fully supported by the stem the high pressure port is closed and the low pressure port open thus preventing hydrostatic pressure build-up on the seal. As a result, little or no seal friction exists. When the closing gate comes to rest on the gate stops, the short over-travel of the stem reverses the action of the valve ports and reservoir pressure is admitted to the seal chamber, effectively forcing the seal in contact with the seal seat. However, the installation increases the cost of the gate and hoist controls and is used only when deemed necessary.

Gate Frames

Gate frames consist of the tracks, track bases, seal seats and bases and the guides. Frames for penstock gates usually have the tracks and vertical seal seats mounted on a common base. Anchor bolts are set in the first stage concrete and project into blockouts provided in the first stage concrete for the track bases, seal seats and guides. After the frames are accurately alined by means of the anchor bolts, the blockouts are filled with concrete.

The wheel loads are transmitted to the concrete structure through the tracks and track bases. The deflection of the horizontal beams from bending due to the load on the gate will tend to rotate the vertical girders at the ends of the gate. The wheels will be rotated with the vertical girders, and in order to prevent overstressing of the edges of the wheel treads and tracks, the track surface is finished with a transverse crown to compensate for the lateral rotation. The radius of the track crown is made sufficiently short to produce a rise of from 1/32 inch to 1/16 inch, depending on the width of the track

and amount of deflection and rotation anticipated. The crown is made on the track instead of on the wheel tread so that there will be no tendency for the wheels to travel as a cone on the track, a condition which would make them travel on the flat track toward the center of the gate. As the wheels on opposite sides of the gate tend to travel toward each other, heavy opposing sliding friction would result between the wheel treads and the track face or (from side thrust) between the wheel hubs and vertical girders, contributing to stalling forces acting on the closing gate. Tracks are usually inaccessible after installation and are therefore made with a Brinnell Hardness Number of 50 points or more higher than the B.H.N. specified for the wheel treads. This tends to confine the wear to the wheels, which are more readily accessible for repair or replacement. Stainless steel type 410 or 416 (heat treated) with minimum ultimate tensile strength of 120,000 psi and minimum yield point in tension of 90,000 psi with B.H.N. 302 minimum is used for the tracks.

The design of the track and its base is based on application of data from tests and theoretical treatises relating to beams on elastic foundations. Only the maximum wheel load is considered, as the track and track base each have constant cross-sections for their loaded length. The track is assumed to act as an infinite beam on a two-dimensional foundation (the track base) and the maximum bending moment [Ref. 9, (20) & (24)] is

$$M_{max} = 0.385\ P \left[\frac{E\ I_t}{EC} \right]^{1/3}$$

which reduces to $M_{max} = 0.212\ Ph$

for a track and track base having the same Modulus of Elasticity and a rectangular track having height h and width 2c. See FIG. XI.

The section modulus of the rectangular track $S_t = (2c)\ h^2/6$ and the track bending stress

$$f_t = \frac{M}{S_t} = \frac{(0.212\ Ph)\ 6}{(2c)\ h^2} \quad \text{and} \quad (2c)h = \frac{1.27\ P}{f_t}$$

A maximum bending stress f_t of 25,000 psi is allowed for track material with minimum 90,000 psi Y.P. and the track area required is

$$A_t = (2c)\ h = 5.08\ (10^{-5})\ P \text{ square inches.}$$

where

M_{max} = Maximum bending moment of track and track base

S_t = Section Modulus of the track

P = Maximum wheel load in pounds

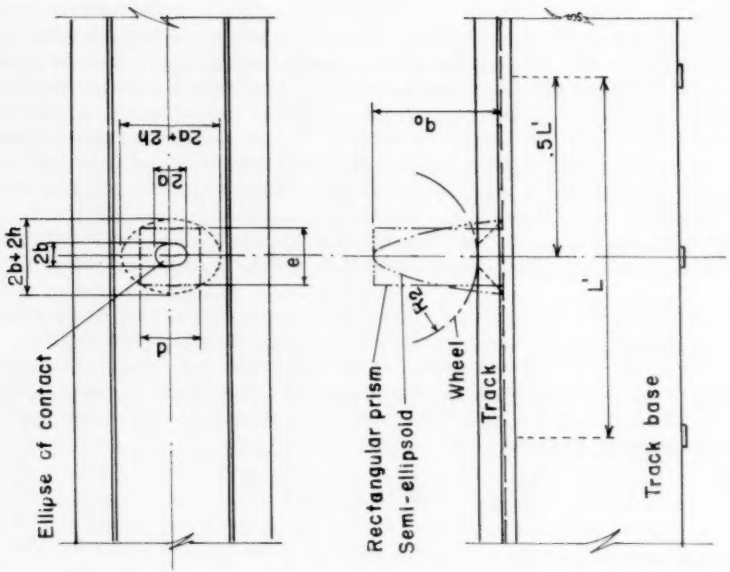
c = One-half the track width, inches

h = Track height, inches

E = Modulus of Elasticity of steel = $30\ (10^6)$

I_t = Moment of inertia of the track

A_t = Track area, square inches



Track and Track Base

FIG. XI

The area of contact between the wheel and track is an ellipse with major axis 2a and minor axis 2b. Values of $\wedge P/a^3$, z_1/a and k are taken from Fig. X and (from Ref. 7) the axes of the ellipse are

$$2a = 2\sqrt[3]{\frac{1.250\ (10^{-7})\ R_1R_2P}{C_1\ (R_1 + R_2)}} \quad \text{where} \quad C_1 = \frac{\wedge P}{a^3}$$

$$2b = 2ka = k(2a) \quad \text{where} \quad k = b/a$$

It is assumed that the cross-section of the track is proportional to the major axis of the ellipse of contact and the depth of maximum shear stress z_1, using a safety factor of 4 for the track depth, then

$$\frac{2c}{h} = \frac{2a}{4z_1}, \quad \text{combine} \quad h = \frac{A_t}{2c}$$

$$\text{track width} \quad 2c = \sqrt{\frac{2a\ A_t}{4\ z_1}}\ ; \quad \text{track height} \quad h = \sqrt{\frac{4z_1\ A_t}{2a}}$$

The wheel load is assumed to be transferred through the track onto the track base by the frustrum of an ellipsoid whose small base is the ellipse of contact between the wheel and track and whose large base at the bottom of the track is a 45° projection of the periphery of the ellipse of contact, the altitude of the frustrum being the track height. The pressure on the ellipse at the bottom of the track is equal to the wheel load, is maximum at the center and diminishes to zero at the edges, forming a semi-ellipsoid of pressure distribution. The maximum intensity of pressure (q_o) at the center is 1-1/2 times the average pressure on the bottom ellipse (Ref. 10) with major axis (2a + 2h) and minor axis (2b + 2h). See Fig. XI.

$$q_o = \frac{3}{2}\ \frac{P}{\pi(a + h)(b + h)}$$

For the design of the track base a rectangular prism (with base area A_r) is substituted for the semi-ellipsoid, and the pressure q_o is assumed to be uniformly distributed over the base of the prism; the total load is equal to the wheel load as above,

$$A_r\ q_o = P \quad \text{or} \quad A_r = \frac{2}{3}\pi(a + h)(b + h)$$

The sides of the prism base d and e are proportional to the axes of the base of the semi-ellipsoid;

$$\frac{d}{e} = \frac{2a + 2h}{2b + 2h} \qquad d = \left(\frac{2a + 2h}{2b + 2h}\right)e \qquad e = \sqrt{A_r \Big/ \frac{2a + 2h}{2b + 2h}}$$

Note:

If the track width 2c previously determined is less than d it must be increased to a minimum width of d to obtain adequate distribution of the wheel load to the track base.

The "tee section" track base design procedure is similar to the procedure used in column base plate design (Ref. 11) for distributing a column load over a sufficient area to satisfy the foundation bearing pressure limitation. While the column base is a flat plate free on all sides, the tee track base flange is free on two sides but restrained on the other two by the stem. The bending stresses of the net flange alone, about the longitudinal axis, determines the effective width W' of the flange, W' = d + 2j where j is the effective cantilever projection of the flange loaded to the permissible concrete bearing stress P' = 1125 psi (for the 3000 pound ultimate strength concrete generally used). Where t' is the net finished thickness of a trail tee section flange

$$(t')^2 = P' (j)^2 / 6000 \text{ and } j = 2.31\, t'$$

The required effective length L' of the tee base is determined by the minimum concrete bearing area requirements:

$$L' = P/1125\, W'$$

The track is not infinitely rigid, along the longitudinal axis, compared to the track base; and the dimension e of the rectangular prism pressure base is small compared to the required effective length L' of the tee base. Therefore the cantilever projection of the tee track base along the longitudinal axis is taken as .5 L'. With an allowable bending stress of 18,000 psi the required section modulus of the track base about the transverse axis is:

$$S = M/18000 = P' (.5L)^2 W'/36000 = 0.00782 (L')^2 W'$$

In order to avoid overlapping the effect of adjacent wheel loads on the track base and foundation stresses the length L' is not allowed to exceed the minimum wheel spacing. This is accomplished by selecting a tee section with a thick flange, thereby obtaining a wide effective W' and a minimum length L'.

Shear stress in the track base is determined by using Ref. 12 (Pg. 16) as a basis. The load is assumed distributed uniformly along the length e of the prism base, and the load per unit length q' = P/e. The maximum shear V occurs equally at the ends of the loaded length e,

$$V = \frac{q'}{4\lambda}\left(1 - C'_{\lambda e}\right) \quad \text{and} \quad \lambda = \sqrt[4]{\frac{k}{4\, E\, I_b}}$$

Average shear $v = V/h_b t_b$ (allowable 11,000 psi for embedded materials)

The longitudinal shear L.S. in the track base web is maximum at the neutral axis

$$L.S. = \frac{VM_s}{I_b t_b} \quad \text{(maximum allowable 11,000 psi)}$$

where

λ = function of constant factors affecting deflection

C' = function of λe (see table 2)

k = foundation modulus

E = modulus of elasticity for steel = 30 (10^6)

E_c = modulus of elasticity for concrete = 3 (10^6)

C'' = factor based on pressure distribution across track base = 1 for assumed uniform distribution.

u = Poisson's ratio for foundation = 0.20 for concrete

I_b = moment of inertia of track base

M_s = statical moment of track base

h_b = net height of tee

t_b = tee web thickness

$$k = 1.29 \left[\frac{1}{C''(1 - u^2)} \frac{E_c \left(\frac{W'}{2}\right)^4}{E I_b} \right]^{0.11} \frac{E_c}{C'' (1 - u^2)} \quad \text{[Ref. 9 (39)]}$$

with constants substituted

$$k = 2.32 (10^6) \left[\frac{(W')^4}{I_b} \right]^{0.11}$$

The transverse bending stress in the track base is checked from the maximum moment occurring under the load; $M = P/4\lambda$ (Ref. 12 Pg. 11) from which the stress $f_b = M/S$ (section modulus of the track base).

The track and track base are designed as independent units; however, at any loaded point they deflect an equal amount and the deflection $y_o = P\lambda/2k$ (Ref. 12 Pg. 11) where λ and k are functions of the combined I of the track and track base (as the I of the track is small compared to the I of the track base, the values of λ and k for the track base alone are sufficiently accurate to determine the deflection).

From the common deflection y_o, a value of λ for the track is obtained by expressing k in terms of λ in the deflection equation above:

$$k = (\lambda^4)\ 4\ EI_t \quad \text{and} \quad y_o = P/8\lambda^3\ EI_t$$

then

$$\lambda_{track} = \sqrt[3]{\frac{P}{8\ y_o\ EI_t}}$$

The maximum moment in the track $M_t = P/r\lambda$track from which the stress in the track is checked

$$f_t = M_t/S_t$$

If the stresses, resulting from bending due to deflection of the track, exceed the allowable, then the track or track base is stiffened.

The tracks, seal seats and guides are submerged and not readily accessible for maintenance or replacement. The plates and bars or angles of these members, that have exposed surfaces subjected to rolling or sliding contact with the wheels and seals or guide shoes on the gate, are made of corrosion-resisting material (usually stainless steel) to reduce corrosion of the contact surfaces. The screws, dowels and bolts used for fastening these parts to their bases are specified to be made of comparable material to diminish galvanic corrosion tendencies. Other parts are made of ordinary structural grade carbon steel.

TABLE 2
(See Table 1, Ref. 6)

FUNCTIONS OF λe AND $C'\lambda e$					
λe	$C'\lambda e$	λe	$C'\lambda e$	λe	$C'\lambda e$
0.25	0.5619	0.35	0.4204	0.45	0.2968
0.26	0.5469	0.36	0.4072	0.46	0.2853
0.27	0.5321	0.37	0.3943	0.47	0.2742
0.28	0.5175	0.38	0.3815	0.48	0.2632
0.29	0.5030	0.39	0.3688	0.49	0.2522
0.30	0.4888	0.40	0.3564	0.50	0.2414
0.31	0.4748	0.41	0.3441	0.51	0.2307
0.32	0.4609	0.42	0.3320	0.52	0.2204
0.33	0.4472	0.43	0.3201	0.53	0.2103
0.34	0.4337	0.44	0.3084	0.54	0.2002

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THE REGULATION CHART FOR POWER COMPUTATIONS[a]

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(Proc. Paper 1421)

SYNOPSIS

In this paper the regulation chart method of computing regulated prime flow and power output from reservoirs and hydro-electric plants is presented. The principles of the preparation of the regulation chart and of its use are given, and its application is illustrated by examples. It is especially useful when a complicated system of reservoirs is being investigated. By the usual procedures the time required to obtain other than an approximation of the prime flows and outputs for such a system would be almost prohibitive. With the regulation charts, correct or essentially correct results can be found quickly. Since the computations are not time consuming, many drawdowns and heights of dam may be tested. The charts are useful also in ascertaining the effect of proposed reservoirs on existing downstream plants. In addition, the paper presents methods of determining the economic limit of installation for secondary power.

INTRODUCTION

When a system of reservoirs and hydro-electric power plants in a river basin, either proposed or existing in part, is to be studied, the determination of the regulated flow and the power output at each plant by traditional methods is a time consuming procedure. When the reservoirs are so located that they affect the flow at other reservoirs, the work is extremely lengthy and laborious. The flows at the upstream site are first regulated with the storage at that site. Next, they are added to the flows from the area intermediate between the upstream site and the next downstream reservoir. The

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resulting flows are then re-regulated with the storage available at the latter reservoir. The procedure is repeated from one reservoir to another until the regulated flow at the farthest downstream reservoir is obtained. From these regulated flows, the primary hydro-electric capacity (the capacity obtained from the minimum regulated flow), the power output, and the installed capacity are determined. When the entire hydro-electric resources of a river basin are being investigated, involving numerous reservoirs and power plants arranged in a complicated system, the time required is almost prohibitive. In order to speed the determination of the output and capacity of such a system of plants and reservoirs, the Regulation Chart and the methods of its use were developed and improved.

The Regulation Chart

The Regulation Chart consists of a series of Storage-Draft curves and a family of Prime Flow-Water Capacity-KWH Output Curves. Examples are shown on Figures 3 and 4. It is sometimes desirable to add or substitute a replotting of the power output curves as shown in Fig. 5.

A separate regulation chart is prepared for each area that has reasonably uniform flow characteristics so the flow expressed in cubic feet per second per square mile of drainage area can be applied without appreciable error to the entire area or any portion of the area for which each chart has been prepared. It should be noted that the regulation charts are plotted in terms of flow, storage, and output per square mile of drainage area. In addition, the output is based upon one foot of head so the total output can be ascertained when the drainage area and the head are known. When one area for which a chart is prepared lies downstream from another, as Area "B" on Figure 2, the chart is prepared using only the flows from Area "B" and the drainage area in Area "B." This is the intermediate area between Stream Gage "X" and Stream Gage "Y."

Preparation of the Storage-Draft Curve

The storage-draft curve shows simply the storages required to maintain flows in the most critical low flow period of record to the rate indicated by the curve. If it were known that one period was most severe for all storage quantities and for all areas for which regulation charts are prepared, only one storage-draft curve would be required. Reference to Figures 3 and 4 will show that for Areas "A" and "B" the critical periods varied with storage and from one area to the other. Accordingly, 4 storage-draft curves were plotted for each area.

The storage-draft curves may be obtained from the monthly flow mass curve, the monthly flow hydrograph, or directly from the stream flow records expressed as average monthly flows. Portions of the mass curve and the hydrograph for Area "A" are shown on Figure 1. The flow records used in this paper are presented in Tables 3 and 4. The storage draft curves are drawn as a series of tangents; and figures, indicating the number of months from the beginning of reservoir drawdown to maximum drawdown, are shown on the tangents. That the curves should be drawn with tangents when monthly flows are used is evident from inspection of the hydrograph (Fig. 1) or the flow records. For example, the minimum flow in 1917 was 0.11 sec. feet per square mile and it occurred in October. For each .01 second foot per square

mile in excess of this, until the length of the period to maximum drawdown changes, .01 sec. foot months of storage are required. This relation represents a straight line. Thus, the storage-draft curve can be easily computed from the records as shown in Table 1 (See Table 3 for the records used.).

TABLE 1

Storage-Draft Curve - Area "A"
1916-1917 Low Flow Period

Draft Sec. Ft. per Sq. Mi.	Months to Max. Drawdown	Increment of Storage Sec. Ft. Months per Sq. Mi.	Period from Beginning of Drawdown to Max. Drawdown	Total Storage Required	
				Sec Ft Months Per Sq Mi	Acre Ft Per Sq Mi.
0.11				0	0
	1	0.13	Oct. 1917		
0.24				0.13	7.84
	3	0.03	Aug. to Oct. 1917		
0.25				0.16	9.65
	4	0.24	Aug. to Nov. 1917		
0.31				0.40	24.1
	5	1.55	Aug. to Dec. 1917		
0.62				1.95	118
	6	1.26	July to Dec. 1917		
0.83				3.21	193
	18	1.26	July 1916-Dec. 1917		
0.90				4.47	270

At a flow of 0.83 sec. ft. per sq. mile the beginning of the drawdown period changed from July 1917 to July 1916. From the mass curve on Figure 1 it can be seen that this occurred when the regulated flow in the 1917 low flow period reached a magnitude equal to the average flow of the preceding high and low flow periods, Point "A" to Point "B." Had the storage been maximum in the 1916 low water period, the point of maximum drawdown would shift from Point "C" to Point "D" when the regulated flow attained a rate equal to the average flow between these two points, that is, when the regulated flow in 1916 equaled the average of the succeeding high and low water periods. The knowledge of these two simple facts is helpful in preparing the storage-draft curves from the stream flow records.

Preparation of the Output Curves

In preparing the Prime Flow-Water Capacity-Output Curves, it has been assumed that until the reservoir is filled once more, only the prime flow is discharged during every low water period in which the natural flow is less than the prime. This method of regulation is shown on the mass curve of Fig. 1 for a prime of 0.60 sec. ft. per sq. mi. This is a conservative method, since by means of rule curve operation, it may be possible to exceed this draft in some low water periods or portions thereof. To obtain the data from the mass curve with which to compute and plot the output curves, several rates of flow are laid off in each low water period by means of a parallel rule. Where each of these intersects the mass curve, showing the reservoir depletion to be refilled, several rates of draft, such as 2, 3, and 4 times the prime flow, are drawn. The spill is measured from these water capacity lines as

SFM = Second Foot Months
W.C. = Water Capacity
KWH in Spill and Outputs are per foot of head.

TABLE 2

Computation of Prime Flow-Water Capacity-KWH Curves

$KWH/yr \text{ in Spill} = \frac{\text{Spill in SFM/yr} \times 730}{14.6}$

$= \text{Spill in SFM/yr} \times 50$

$KW = \frac{QH}{14.6}$

Efficiency = 80.8%

Output, Area "A" = 637.9 - Spill in KWH/yr

AREA "A"

Prime Flow Sec. Ft. per sq. mi.	W.C. = Prime Flow			W.C. = 2 x Prime			W.C. = 3 x Prime			W.C. = 4 x Prime		
	Spill/yr		Output KWH/sq mi per yr	Spill/yr		Output KWH/sq mi per yr	Spill/yr		Output KWH/sq mi per yr	Spill/yr		Output KWH/sq mi per yr
	SFM/sq mi	KWH/sq mi		SFM/sq mi	KWH/sq mi		SFM/sq mi	KWH/sq mi		SFM/sq mi	KWH/sq mi	
0.20	10.358	517.9	120.0	8.619	430.95	206.95	7.189	359.45	278.45	5.973	298.65	339.25
0.40	7.958	397.9	240.0	5.561	278.05	359.85	3.919	195.95	441.95	2.877	143.85	494.05
0.60	5.558	277.9	360.0	3.521	176.05	461.85	2.220	111.0	526.90	1.389	69.45	568.45
0.80	3.158	157.9	480.0	1.810	90.5	547.4	1.049	52.45	585.45	0.643	32.15	605.75
0.90	1.958	97.9	540.0	1.000	50.0	587.9	0.404	20.2	617.7	0.192	9.6	628.3

AREA "B"

Output, Area "B" = 639.65 - Spill in KWH/yr

0.20	10.393	519.65	120.0	8.617	430.85	208.8	7.161	358.05	281.60	5.982	299.1	340.55
0.40	7.993	399.65	240.0	5.398	269.9	369.75	3.767	188.35	451.3	2.723	136.15	503.5
0.60	5.593	279.65	360.0	3.360	168.0	471.65	2.057	102.85	536.8	1.239	61.95	577.7
0.80	3.193	159.65	480.0	1.786	89.3	550.35	1.066	53.3	586.35	0.620	31.0	608.65
0.90	1.993	99.65	540.0	1.134	56.7	582.95	0.640	32.0	607.65	0.355	17.75	621.9

shown on Fig. 1 for the 0.60 cfs prime flow and the 1.20 cfs water capacity. These are added to give the total spill in the period of record. The spill can be computed directly from the flow records as was done for this paper. To facilitate the calculation somewhat it was assumed that water was released at the water capacity rates, when the stream flow equaled or exceeded those rates, starting on the first of the month in which the reservoir would fill if water were released only at the prime rate. This results in slightly less spill than the assumption that the reservoir is completely filled before the prime rate of flow is exceeded. However, the difference is well within the accuracy of the records, and the method may be more nearly the correct one when the conservative regulation procedure is considered.

Table 2 shows the assembling of the data obtained as described above and the computation of the Prime Flow-Water Capacity-Output Curves. The total potential outputs were computed from the stream flow records as shown on Tables 3 and 4. The derived outputs have been plotted on the regulation charts, Figures 3 and 4, and also for Area "A" on Fig. 5. The latter employs the prime flow as the parameter whereas the curves on the regulation charts employ the water capacity of the installation as the parameter.

Use of the Regulation Chart

Figure 2 is a map of an assumed drainage area with the location of potential reservoirs shown thereon. The flow and power characteristics of Area "A" are represented by Figure 3 and of Area "B" by Figure 4.

Before proceeding with the calculations, there are two basic rules that must be mastered in order that the correct regulated prime flows may be obtained from the charts.

Rule No. 1

When use of the storage in a downstream reservoir results in a prime flow from the area between an upstream reservoir and the downstream reservoir that persists, to the time of maximum drawdown, for a lesser number of months than the storage at the upstream reservoir causes the prime flow there to persist, the two individual prime flows are simply added to obtain the total prime flow at the lower reservoir.

Rule No. 2

When the storage at a downstream reservoir applied to the storage-draft curve of the intermediate area between the two reservoirs causes the prime flow to persist, to the time of maximum drawdown, for a greater number of months than the storage at the upstream reservoir causes the prime flow there to persist, the storages in both reservoirs are added. The total is treated as though it were all located at the downstream reservoir. The prime flow obtained therefrom is the prime flow at the downstream reservoir.

Rule No. 1 is based on the principle that if the flow from the drainage area above an upstream reservoir is regulated by that reservoir to a higher unit prime flow than that to which the flow from the area between that reservoir and the downstream reservoir is regulated by the lower reservoir, the two regulated prime flows generally may be added to obtain the total prime flow at the downstream reservoir. Thus the lower reservoir simply regulates the flow from the intermediate area. Since the intermediate area is regulated to

TABLE 3

FLOW - AREA "A"

Sec. Feet per Sq. Mile of Drainage Area in Area "A"

Months	1909	1910	1911	1912	1913	1914	1915	1916	1917	1918	1919
Jan.		0.72	1.21	1.26	0.98	0.84	1.04	0.71	2.00	1.16	1.87
Feb.		1.17	1.91	1.17	1.70	1.81	0.70	1.94	1.05	5.29	2.90
Mar.		0.90	1.03	1.00	2.97	2.24	1.30	4.35	1.14	1.65	5.52
Apr.		1.73	1.05	1.56	1.54	0.95	1.11	2.18	1.52	2.82	2.92
May	3.33	0.89	0.57	0.45	2.15	1.01	1.07	7.18	0.94	0.51	
June	0.75	1.14	2.74	0.38	0.43	0.69	0.56	2.12	0.88	0.68	
July	0.46	0.22	0.57	0.22	0.44	0.46	0.26	0.75	0.62	0.14	
Aug.	0.40	0.15	2.07	0.25	0.41	0.22	0.16	0.60	0.16	0.36	
Sept.	0.26	0.14	0.18	1.31	0.38	0.18	0.30	0.28	0.32	2.85	
Oct.	0.29	0.10	0.16	0.88	0.18	0.13	0.39	0.25	0.11	1.38	
Nov.	0.28	0.11	0.15	0.62	0.46	0.10	0.11	0.20	0.25	0.93	
Dec.	0.25	0.22	0.22	1.06	1.17	0.24	0.21	0.35	0.31	1.97	
Totals	6.02	7.49	11.86	10.16	12.81	8.87	7.21	20.91	9.30	19.74	13.21
Σ Totals	6.02	13.51	25.37	35.53	48.34	57.21	64.42	85.33	94.63	114.37	127.58

Potential KWH/ft. of head/sq. mi./yr = $\frac{127.58}{10} \times \frac{1}{14.6} \times 730 = 637.9$

TABLE 4

FLOW - AREA "B"

Sec. Feet per Sq. Mile of Drainage Area in Area "B"

Months	1909	1910	1911	1912	1913	1914	1915	1916	1917	1918	1919
Jan.		1.00	1.25	1.31	0.99	0.88	1.09	0.80	1.50	1.02	1.80
Feb.		1.25	2.03	1.23	1.65	1.92	0.77	2.01	0.95	4.78	3.00
Mar.		1.00	1.14	1.10	2.80	2.33	1.40	3.31	1.00	1.80	5.56
Apr.		2.00	0.97	1.40	1.60	0.97	1.21	2.50	1.40	3.01	2.85
May	3.30	0.90	0.62	0.44	2.16	1.07	1.11	6.94	0.90	0.62	
June	0.80	1.25	3.00	0.35	0.40	0.75	0.61	2.04	0.81	0.70	
July	0.40	0.23	0.64	0.24	0.44	0.60	0.30	0.80	0.49	0.20	
Aug.	0.50	0.22	1.21	0.31	0.39	0.35	0.20	0.66	0.13	0.25	
Sept.	0.30	0.20	0.44	1.50	0.37	0.22	0.30	0.21	0.18	2.80	
Oct.	0.25	0.21	0.31	0.90	0.15	0.15	0.40	0.15	0.10	1.30	
Nov.	0.23	0.20	0.22	0.71	0.50	0.20	0.25	0.10	0.13	0.90	
Dec.	0.21	0.24	0.28	1.16	1.23	0.29	0.28	0.30	0.15	2.00	
Total	5.99	8.70	12.11	10.65	12.68	9.73	7.92	19.82	7.74	19.38	13.21
Σ Totals	5.99	14.69	26.80	37.45	50.13	59.86	67.78	87.60	95.34	114.72	127.93

Potential KWH/ft. of head/sq. mi./yr = $\frac{127.93}{10} \times \frac{1}{14.6} \times 730 = 639.65$

a lesser degree than the area above the upper reservoir, reference to the storage-draft curves will show that application of the entire storage in the lower reservoir to the flows of the intermediate area is usually more productive of prime flow than would be the case were a part of that storage applied to the flows from the upper reservoir which are already more highly regulated than the flows from the intermediate area. However, even if the regulated unit prime flow from the intermediate area is less than the regulated unit prime flow from the area above the upstream reservoir, should the regulated prime flow on each area to the time of maximum drawdown persist for the same number of months, the correct total prime flow will be obtained at the lower reservoir, if part of the storage in that reservoir is applied to the upper area or if part of the upper reservoir storage is applied to the intermediate area, so long as the number of months to maximum drawdown is not changed for either area. This follows from the fact that as long as the prime flow persists for a given number of months to the time of maximum drawdown, an equal increment of storage will produce an equal increment of prime flow from either area, for the slope of the storage draft curve is the same for all areas and is constant as long as the number of months is the same.

Rule No. 2 is based upon the principle that if sufficient storage is available in the lower reservoir to raise the low flows from the intermediate area to the same unit prime flows as those produced by the upper area any additional storage in the lower area will furnish additional regulation to flows from both areas. Stated in this manner the assumption is made that the number of months from the beginning of drawdown to maximum drawdown is the same for both areas. If the number of months is not the same, the additional storage is divided so the number of months is the same. As in the case of Rule No. 1, the available storage is thereby used most effectively. It follows from this that when storage in the lower reservoir is capable of regulating the flows from the intermediate area to a higher degree than storage in the upper reservoir is capable of regulating the flows from the area above it, the total prime flow at the lower reservoir may usually be obtained by adding the storages in both reservoirs and applying the sum to the unregulated flows from the entire drainage area above the lower reservoir. However, as the storage-draft curve is a series of tangents, the slope of each representative of the number of months from beginning of prime draft in the critical low water period to the time of maximum drawdown, the same operation will produce the same prime flow at the lower reservoir if the storage in the lower reservoir when applied to the intermediate area flows would cause the regulated flow from that area to persist for a greater or the same number of months as the storage in the upper reservoir causes the regulated flow from the area above it to persist. This results from the fact that the storage needs only to be most effectively used.

Both Rules No. 1 and No. 2 are expressed in number of months to maximum drawdown because they are most useful in that form when various areas, the flows from which must be regulated by one or more reservoirs, have differing flow characteristics.

Computation of the Prime Flow

In order to demonstrate the use of storage-draft curves and the application of the foregoing rules in determining the prime flow, various

combinations of reservoirs shown on Figure 2 will be employed. The drainage areas are as follows:

Area "A"	-	500 sq. miles
Area "B"	-	900 sq. miles
Reservoir No. 1	-	300 sq. miles
Reservoir No. 2	-	500 sq. miles
Reservoir No. 3	-	250 sq. miles
Reservoir No. 4	-	1350 sq. miles
Area between Reservoirs No. 1 and No. 2	-	200 sq. miles
Area between Reservoirs No. 2, No. 3, and No. 4	-	600 sq. miles.

The flow characteristics of Area "A" and Area "B" differ. The flows are given in Tables 3 and 4. Figure 3 is the Regulation Chart for Area "A," and Figure 4 is the Regulation Chart for Area "B."

Example No. 1: Rule No. 1 applies; same flow characteristics at all reservoirs.

Assume a system consisting only of Reservoirs No. 1 and No. 2 (Fig. 2) with storages as follows:

Reservoir No. 1 - 90,000 acre feet = 300 acre ft./sq. mile.
Reservoir No. 2 - 30,000 acre feet = 150 acre ft./sq. mile of the intermediate area.

Since both reservoirs are located in Area "A" only the regulation chart for Area "A" (Fig. 3) need be used. From the storage-draft curves the following information is obtained:

Reservoir No. 1 -
Regulated Flow = .746 cfs/sq. mi. = 224 cfs
Critical Period - 1914-1916
Months to Maximum Drawdown - 20
Reservoir No. 2 -
Regulated Flow on Intermediate Area = .574 cfs/sq. mi.
= 115 cfs
Critical Period - 1909-1910
Months to Maximum Drawdown - 6

Since the number of months to maximum drawdown is less on the lower area than on the upper area, Rule No. 1 applies and the prime flows are simply added to obtain the total prime flow at Reservoir No. 2. Thus, prime flow at Reservoir No. 2 = 224 cfs + 115 cfs = 339 cfs.

Note that when the storage at Reservoir No. 2 was applied to the intermediate drainage area between the reservoirs, the critical period was different than the critical period for Reservoir No. 1. Thus, in the 1914-1916 period, the storage in Reservoir No. 2 was not depleted and in the 1909-1910 period the storage in Reservoir No. 1 was not depleted. Reference to the storage draft curves will show that had both reservoirs been operated at all times to secure the maximum prime flow at Reservoir No. 2, the minimum flows at that reservoir would have been as follows:

1909-1910 Period - $0.798 \times 300 + 0.574 \times 200 = 354$ cfs
1914-1916 Period - $0.746 \times 300 + 0.613 \times 200 = 347$ cfs

By this method of regulation the prime flow would be 347 cfs. However, as previously mentioned, for this paper the more conservative regulation procedure was assumed whereby each reservoir discharges only its prime

flow in all low water periods in which the inflow is less than the prime flow. By this method of regulation the prime flow at Reservoir No. 2 is 339 cfs as given above.

Example No. 2: Rule No. 2 applies; same flow characteristics at all reservoirs.

Assume the following storages in Reservoirs No. 1 and No. 2:

Reservoir No. 1 - 30,000 Acre Feet = 100 Acre Feet per sq. mile.

Reservoir No. 2 = 50,000 Acre Feet = 250 Acre Feet per sq. mile of intermediate area.

Since both reservoirs are in Area "A" the storage-draft curves on Figure 3 apply to both. Reference to these curves will show that with the above storages the period to maximum drawdown will be 19 months for Reservoir No. 2 and 6 months for Reservoir No. 1. Accordingly, Rule 2 applies and the regulated prime flow from Reservoir No. 2 is obtained by adding the storages available at the two reservoirs and applying the total to the entire drainage area above Reservoir No. 2.

Then, Total Storage = 80,000 Acre Ft. on 500 sq. miles of drainage area

$$= \frac{80{,}000}{500} = 160 \text{ Acre Ft./sq. mile}$$

Prime Flow at Reservoir No. 2 (Fig. 3) = 0.60 cfs/sq. mile

= 0.60 x 500 = 300 csf

Since for the above storage the critical low flow period was the 1909-1910 period, the same as for Reservoir No. 1, no adjustment for varying critical periods is involved.

Example No. 3: Rule No. 1 applies; flow characteristics differing in two areas.

Assume a system of reservoirs consisting of Reservoirs No. 1 and No. 2 with the same storages and regulation as given in Example No. 2, and Reservoir No. 4 with 85,000 acre feet of storage or 100 acre feet per square mile on 850 square miles, the portion of Area "B" above Reservoir No. 4, (see Fig. 2). The flow characteristics of Area "A" are represented by Figure 3 and of Area "B" by Figure 4. With the 160 acre feet per square mile effective at Reservoir No. 2 (total storage in Reservoirs No. 1 and No. 2) and 100 acre feet per square mile on the intermediate area at Reservoir No. 4, the following information is obtained from the Regulation Charts:

Reservoir No. 2 (Fig. 3) -

No. of months to maximum drawdown - 6

Critical Period - 1909-1910

Reservoir No. 4 (Fig. 4) -

No. of months to maximum drawdown - 5

Critical Period - 1916-1917

As the number of months to maximum drawdown is less at Reservoir No. 4 than at Reservoir No. 3, the effective storage at Reservoir No. 2 is applied to the flows from the area above that reservoir and the storage at Reservoir No. 4 is applied to the intermediate area. Because of the method of regulation assumed herein, the results are not affected by the differing critical period. The total prime flow at Reservoir No. 4 is obtained from the regulation charts as follows:

Prime at Reservoir No. 2 = 0.60 x 500 sq. mi. = 300 cfs.

Prime from Intermediate Area = .47 x 850 sq. mi. = 400 cfs

Total Prime at Reservoir No. 4 = 700 cfs.

Example No. 4: Rule No. 2 applies; flow characteristics differing in two areas.

Assume the same system of reservoirs as in Example No. 3 except that Reservoir No. 4 has 340,000 acre feet of storage or 400 acre feet per square mile on 850 square miles, the portion of Area "B" between Reservoirs No. 2 and No. 4. From Figures 3 and 4 the following information is obtained:

Reservoir No. 2 - 160 acre ft. per sq. mi. - (Fig. 3)
No. of months to maximum drawdown - 6
Critical Period - 1909-1910
Reservoir No. 4 - 400 acre ft. per sq. mi. - (Fig. 4)
No. of months to maximum drawdown - 32
Critical Period - 1913-1916

As the number of months to maximum drawdown is greater at Reservoir No. 4 than at Reservoir No. 2, the storages are added and applied to the total drainage area above Reservoir No. 4. In this summation, the effective storage at Reservoir No. 2 is used. However, because of the method of regulation used herein whereby each reservoir discharges only its prime flow during each low water period, and because the critical periods differ, the storages may have to be adjusted prior to final determination of the prime flow at Reservoir No. 4. By first adding the storages without adjustment, it will be found that the total storage is 420,000 acre feet equal to 311 acre feet per square mile on the 1350 square miles above Reservoir No. 4. With this storage, Figures 3 and 4 indicate that the critical period for both areas is the 1913-1916 period. But since the regulated flow at Reservoir No. 2 is 0.60 sec. feet per square mile, only 136 acre feet per square mile or 68,000 acre feet of storage is utilized in Reservoirs No. 1 and No. 2 in the 1913-1916 period. Therefore, the total to be employed to regulate flows for Reservoir No. 4 is -

Reservoirs No. 1 and No. 2	=	68,000 acre feet
Reservoir No. 4	=	340,000 acre feet
Total	=	408,000

= 302 acre feet/sq. mile.

With this storage, the number of months to maximum drawdown on both Figures 3 and 4 is shown to be 20 months. When the number of months is not the same, the total storage is arbitrarily reassigned to the areas so that the number of months is the same for both areas. For example, had the total storage been 275 acre feet per square mile or 371,000 acre feet, the number of months to maximum drawdown would have varied. In that case, 285 acre feet per square mile or 242,000 acre feet might have been assigned to Area "B" and the remaining 129,000 acre feet or 258 acre feet per square mile to Area "A." The number of months would then have been the same for both areas.

With the 302 acre feet per square mile, the prime flow at Reservoir No. 4 is found as follows:

Prime from Area "A" (Fig. 3) = .748 x 500	=	374 cfs
Prime from Area "B" (Fig. 4) = .816 x 850	=	694 cfs
Total Prime Flow, Reservoir No. 4	=	1068 cfs

Had the method of regulation adopted been such that the upstream reservoirs would operate to produce the maximum prime flow at the downstream reservoir, it would not be necessary to adjust the storage in Example No. 4 because of differing critical low flow periods. The total storage in the reservoirs, 420,000 acre feet, would be used to obtain the prime flow at Reservoir

No. 4. As noted in Example No. 1, were this method of regulation employed, when the storage in the lower reservoir regulates the flow from the intermediate area to the time of maximum drawdown for a lesser number of months than the regulated prime flow at the upper reservoir, and the critical periods for the two areas differ, the storages in each reservoir are applied to both critical periods. The prime flows obtained from each area are added separately for each critical period. The minimum of these sums is then the prime flow at the downstream reservoir. Thus, in Example No. 1, the prime flow at Reservoir No. 2 for this method of regulation is 347 cfs.

General Procedure when Flow Characteristics Vary above a Reservoir

When Rule No. 1 applies, the procedure is exactly as stated in that rule and illustrated in Example No. 3. Each reservoir regulates the flows from the area above it or the area between it and the upstream reservoirs. The resulting individual prime flows are added to produce the total prime flow at the downstream reservoir.

As illustrated in Example No. 4, when Rule No. 2 applies and two or more areas above the downstream reservoir have differing flow characteristics, the total effective storage at the downstream reservoir is arbitrarily divided between those areas so that the number of months to maximum drawdown is the same for all areas. The prime flows obtained separately for each area by use of the storages so assigned are added to obtain the total prime flow at the downstream reservoir.

If there are no upstream reservoirs and there are two or more areas of differing flow characteristics above a reservoir, the storage is so divided between the areas that the number of months to maximum drawdown is the same for all areas. The regulated flow from each area is then obtained by use of the storage assigned to each and the results are added to produce the prime flow at the reservoir.

It may be noted from the storage draft curves that the number of months to maximum drawdown does not always increase one month at a time as the storage increases. As a result occasionally when the storage is divided between the various areas, there may be no monthly period on the storage-draft curve for one or more of the areas that appears to be the proper period on the storage-draft curves for the other areas. In that case, the storage assigned to the areas lacking this monthly period should be the amount indicated at the cusp of each storage-draft curve where the period passes over the number of months indicated as the proper period. For example, if the storage-draft curve for an area changes from a 19-month period to a 21-month period with no 20-month period between, and the tentative division of storage shows, from the storage-draft curves of the other areas, that the proper period is 20 months, the storage assigned to the area that does not have a 20-month period will be the amount indicated on the storage-draft curve at the junction between the 19-month and 21-month periods.

Computation of the Power Output

To demonstrate the procedure of obtaining the power output from the Regulation Charts, the same examples will be used as for the computation of the prime flow given above. In each instance, the water capacity of the installation will be assumed to be twice the prime flow, equivalent to a 50% load factor installation.

Example No. 1: Rule No. 1 applies; same flow characteristics at all Reservoirs.

Reservoir No. 1 -

Prime Flow = 0.746 cfs/sq. mi.

Avg. Annual Output (Fig. 3) = 525 kwh/foot of head/sq. mile

Annual Primary Output = $\frac{.746}{14.6}$ x 8760 = 600 x .746

= 448 kwh/foot of head/sq. mile

Avg. Annual Secondary Output = 77 kwh/foot of head/sq. mile

Reservoir No. 2 -

The prime flow at Reservoir No. 2 is the prime flow from Reservoir No. 1 added to the prime flow obtained from the intermediate area by application of the storage available in Reservoir No. 2 to the flows from that area. In order to obtain the power output at Reservoir No. 2, the excess flow capacity of the installation above the prime may be applied either to the regulated flows from the area above Reservoir No. 1, to the regulated flows from the intermediate area, or partially to both. To obtain the correct power output, the excess capacity must be applied so as to obtain the maximum power output from the Regulation Chart. This is done in the following manner:

Prime Flow, Reservoir No. 1	=	224 cfs
Prime Flow, Intermediate Area	=	115 cfs
Prime Flow, Reservoir No. 2	=	339 cfs
Water Capacity of Installation	=	2 x Prime
	=	678 cfs
Excess Capacity over Prime Flow	=	339 cfs

First Trial

Since the intermediate area is regulated to a lesser degree (the number of months to maximum drawdown is less) than the area above Reservoir No. 1, it appears reasonable to assume that application of a substantial portion of the excess flow capacity to the intermediate will be more productive of power output than were it applied to the area above Reservoir No. 1. Assume first that a sufficient portion of the excess capacity is applied to the area above Reservoir No. 1 so the water capacity assigned to this area is 1.4 times the prime flow. To obtain the output at Reservoir No. 2 for this assumption, the following information is needed:

Prime Flow Reservoir No. 1	=	.746 cfs/sq. mile
Prime Flow from Intermediate Area	=	.574 cfs/sq. mile
Water Capacity for Area above Reservoir No. 1	=	1.4
x Prime Flow = 1.4 x .746 = 1.044 cfs/sq. mi.	=	314 cfs
Total Water Capacity at Reservoir No. 2	=	678 cfs
Water Capacity for Intermediate Area	=	364 cfs

= $\frac{364}{200}$ = 1.82 cfs/sq. mile

= $\frac{364}{115}$ = 3.17 x Prime Flow.

For the purpose of obtaining the output the Prime Flow-Water Capacity-Output Curves plotted as shown on Figure No. 5 are more accurate than those shown on the regulation charts because good interpolation is difficult on the regulation chart output curves.

Output from Area above Reservoir No. 1 (Fig. 5) -
= 487 x 300 sq. miles = 146,100 kwh/ft. of head
Output from Intermediate Area (Fig. 5) -
= 527 x 200 sq. miles = 105,400 kwh/ft. of head
Total Output at Reservoir No. 2 = 251,500 kwh/ft. of head

Final Trial

After a few trials the following division of flow capacity was found to produce the maximum output:

Water Capacity for area above Reservoir No. 1 = 1.70 x Prime Flow
= 1.7 x .746 = 1.27 cfs/sq. mi. = 381 cfs
Total Water Capacity at Reservoir No. 2 = 678 cfs
Water Capacity for Intermediate Area = 297 cfs

$= \frac{297}{200} = 1.485$ say 1.49 cfs/sq. mile

$= \frac{297}{115} = 2.58$ x Prime Flow.

Output from Area above Reservoir No. 1 (Fig. 5) =
= 510 x 300 sq. miles = 153,000 kwh/ft. of head
Output from Intermediate Area (Fig. 5) =
= 498 x 200 sq. miles = 99,600 kwh/ft. of head
Total Output at Reservoir No. 2 = 252,600 kwh/ft. of head

Thus, 252,600 kwh per foot of head per year is essentially the correct average annual output for Reservoir No. 2. Then,

Total Average Annual Output, Reservoir No. 2 = 252,600 kwh/ft. of head
Annual Primary Output at Reservoir No. 2 =

$= \frac{339 \text{ cfs}}{14.6} \times 8780$ = 203,400 kwh/ft. of head

Average Annual Secondary Output,
Reservoir No. 2 = 49,200 kwh/ft. of head

For quick calculation it is often not worthwhile to take the time to find the maximum output. Since the primary output is not affected by the calculation, the total output can usually be found with accuracy sufficient for the purpose by assigning the water capacity to each area in accordance with the load factor of the installation or the ratio of the installed water capacity to the prime flow. By this means, the output at Reservoir No. 2 would be obtained as follows:

Water Capacity for Area above Reservoir No. 1 =
= 2 x Prime Flow = 2 x .746 cfs/sq. mi. = 1.492 cfs/sq. mile
Water Capacity for Intermediate Area = 2 x Prime Flow =
= 2 x .574 cfs = 1.148 cfs/sq. mile
Output from Area above Reservoir No. 1 (Fig. 3) =
= 525 x 300 sq. mile = 157,500 kwh/ft. of head
Output from Intermediate Area (Fig. 3) =
= 449 x 200 sq. miles = 89,800
Total Average Annual Output, Reservoir No. 2 = 247,300 kwh/ft. of head
Annual Primary Output at Reservoir No. 2 = 203,400 kwh/ft. of head
Average Annual Secondary Output,
Reservoir No. 2 = 43,900 kwh/ft. of head

Notice that this secondary output is only 5,300 kwh per foot of head smaller than the amount obtained by finding the maximum output. At 3 mills

per kilowatt hour for a plant with a head of 100 feet, the difference in annual value of the two secondary outputs would be only $1590.

Example No. 2: Rule No. 2 applies; same flow characteristics at both reservoirs.

As the output at Reservoir No. 1 is obtained in exactly the same manner as in Example No. 1, the output for that reservoir will not be recalculated. The output for Reservoir No. 2 is obtained as follows:

Prime Flow at Reservoir No. 2 = 0.60 cfs/sq. mile.

Average Annual Output for Water Capacity of 2 x Prime Flow (Fig. 3) = 462 kwh/sq. mi./ft. of head

Annual Prime Output = $\frac{0.60}{14.6}$ x 8760 = 360 kwh/sq. mi./ft. of head

Average Annual Secondary Output = 102 kwh/sq. mi./ft. of head

The foregoing outputs are multiplied by the total drainage area above Reservoir No. 2 (500 square miles) to obtain the outputs per foot of head.

Example No. 3: Rule No. 1 applies; flow characteristics differing in two areas.

Since the outputs at Reservoirs No. 1 and No. 2 would be the same as for Example No. 2, these outputs will not be recalculated for Example No. 3. The following information is pertinent to the determination of the output at Reservoir No. 4.

Prime Flow at Reservoir No. 2 = 0.60 cfs/sq. mile
= 0.60 x 500 sq. miles = 300 cfs

Prime Flow from Intermediate Area =
= 0.47 cfs/sq. mile = 0.47 x 850 sq. miles = 400 cfs

Total Prime Flow at Reservoir No. 4 = 700 cfs

Water Capacity of Installation = 2 x Prime = 1400 cfs

First, the approximate method of determining the output at Reservoir No. 4 will be employed as follows:

Output from Area above Reservoir No. 2 for prime flow of 0.60 cfs/sq. mile and a water capacity of 2 x Prime (Fig. 3) =
= 462 kwh x 500 sq. mi. = 231,000 kwh/ft. of head

Output from Intermediate Area for a prime flow of 0.47 cfs/sq. mile and a water capacity of 2 x Prime (Fig. 4) =
= 408 kwh x 850 sq. miles = 346,800 kwh/ft. of head

Total Average Annual Output = 577,800 kwh/ft. of head

Annual Primary Output = $\frac{700}{14.6}$ x 8760 = 420,000 kwh/ft. of head

Average Annual Secondary Output = 157,800 kwh/ft. of head

Since Rule No. 1 applies the correct output for Reservoir No. 4 will be obtained by assigning the water capacity to the two areas in such a manner as to obtain the maximum output just as was done in Example No. 1 above. After several trials the maximum output was found from curves for Areas "A" and "B" similar to Figure 5, as follows:

Water Capacity for Area above Reservoir No. 2 (Area "A") =
1.6 x Prime Flow = 0.96 cfs/sq. mile = 480 cfs

Total Water Capacity at Reservoir No. 4 = 1400 cfs

Water Capacity for Intermediate Area (Area "B") = 920 cfs
= 2.3 x Prime Flow = 1.082 cfs/sq. mile

Output from Area above Reservoir No. 2 (Area "A") =

= 423 x 500 sq. miles = 211,500 kwh/ft. of head
Output from Intermediate Area (Area "B") =
= 440 x 850 = 374,000 kwh/ft. of head
Average Annual Output, Reservoir No. 4 = 585,500 kwh/ft. of head
Annual Primary Output = 420,000 kwh/ft. of head
Average Annual Secondary Output = 165,000 kwh/ft. of head

Example No. 4: Rule No. 2 applies; flow characteristics differing in two areas.

As Reservoirs No. 1 and 2 are the same as in Examples 2 and 3, the outputs for these reservoirs will not be recalculated. By reference to Example No. 4 of the prime flow calculations given previously, the following information is obtained:

Effective Storage to be applied to entire area above Reservoir No. 4 = 408,000 acre feet = 302 acre feet/square mile on 1350 square miles.
Prime Flow from Area "A" = .748 cfs/sq. mile x 500 = 374 cfs
Prime Flow from Area "B" = .816 cfs/sq. mile x 850 = 694 cfs
Total Prime Flow at Reservoir No. 4 = 1068 cfs
Water Capacity = 2 x Prime Flow = 2136 cfs
Number of Months to Maximum Drawdown - 20

The output at Reservoir No. 4 is first computed by the quick method as follows, using the above prime flows and water capacities equal to 2 x the prime flows:

Output from Area above Reservoir No. 2 (Area "A," Fig. 3) =
= 524 x 500 sq. miles = 262,000 kwh/ft. of head
Output from the Intermediate Area (Area "B," Fig. 4) =
= 555 x 850 sq. miles = 471,750 kwh/ft. of head
Average Annual Output, Reservoir No. 4 = 733,750 kwh/ft. of head

Normally the quick method produces results of sufficient accuracy. The correct output, however, is the maximum that can be obtained without modifying the prime flow at the lower reservoir. It has been shown previously that when Rule No. 2 applies the prime flow at the lower reservoir is not changed by reallocation of the storage to the various areas to any extent that will not cause the number of months to maximum drawdown to change for any one of the areas with differing flow characteristics that are regulated by the effective storage at the downstream reservoir. Thus, the prime flows obtained from the individual areas are changed without changing the total prime flow at the downstream reservoir. In order to determine the maximum output, this is first done using the flow capacities in terms of the same multiple of the prime flow that represents the ratio of the water capacity to the prime at the lower reservoir. When the maximum output by this procedure is found, the prime flows for the individual areas used to obtain this maximum are not changed thereafter. Using these prime flows, the excess water capacity over the prime flow at the lower reservoir is assigned to the various areas by cut-and-try process to obtain the final maximum output in the same manner as demonstrated in Example No. 1 and explained in Example No. 3.

Applying the first step of this procedure, it was found that a prime flow of .775 cfs/sq. mile for Area "A" and a prime flow of .80 cfs/sq. mile for Area "B" produced substantially the maximum output for a water capacity assigned to each area of 2 times the prime flow. The following shows the calculation for this division of prime flow:

Assume storage for Intermediate Area (Area "B") = 283 acre ft./sq. mile
= 240,550 acre feet
Total Effective Storage Available = 408,000 acre feet
Storage for Area "A" = 167,450 acre feet
= 337 acre feet/sq. mile

By reference to Figures 3 and 4, it will be seen that the number of months to maximum drawdown remains at 20. Therefore, this division of storage may be used without changing the prime flow at Reservoir No. 4. Then,

Prime Flow from Area "A" (Fig. 3) = .775 cfs/sq. mile
= .775 x 500 = 388 cfs
Prime Flow from Intermediate Area (Fig. 4) = .80 cfs/sq. mile
= .80 x 850 = 680 cfs
Prime Flow at Reservoir No. 4 = 1068 cfs
Output with Water Capacity = 2 x Prime Flow -
Output from Area "A" (Fig. 3) = 536 x 500 = 268,000 kwh/ft. of head
Output from Intermediate Area (Fig. 4) =
= 550 x 850 = 467,500 kwh/ft. of head
Total Output = 735,500 kwh/ft. of head

When the second step to obtain the maximum output was applied, it was found that the maximum occurred when the water capacity assigned to Area "A" was 2.5 times the above prime flow for Area "A." The final trial is shown below:

Water Capacity for Area "A" = 2.5 x 388 cfs = 970 cfs
Water Capacity at Reservoir No. 4 = 2136 cfs
Water Capacity for Intermediate Area = 1166 cfs
$= \frac{1166}{680} = 1.715 \times$ Prime Flow
Output from Area "A" (Fig. 3) = 557 x 500 = 278,500 kwh/ft. of head
Output from Intermediate Area (Fig. 4) =
= 530 x 850 = 450,500 kwh/ft. of head
Avg. Annual Output, Reservoir No. 4 = 739,000 kwh/ft. of head
Annual Primary Output = 1068 cfs x 600 = 640,800 kwh/ft. of head
Average Annual Secondary Output = 98,200 kwh/ft. of head

Reservoir No. 3 added to System of Example No. 4

If to the system of reservoirs in Example No. 4, Reservoir No. 3 (Fig. 2) is added with a storage that regulates its flow so that the prime persists to time of maximum drawdown for a lesser number of months than the prime persists at Reservoir No. 4 with the flow from the remainder of its drainage area regulated by the effective storage of 408,000 acre feet, Rule No. 2 applies. Then the storage in Reservoir No. 3, or such part of it as is effective in producing the prime flow at Reservoir No. 4, is added to the 408,000 acre feet. The total is used to regulate the flows and obtain the output at Reservoir No. 4 in exactly the same manner as in Example No. 4 above.

Assume instead that Reservoir No. 3 has a sufficient amount of storage to regulate its flow so that the prime persists to the time of maximum drawdown for a greater number of months than the prime from the remainder of the drainage area persists when the flows from the latter area are regulated by the effective storage (408,000 acre feet) at Reservoir No. 4. Thus, Rule No. 1 applies with respect to Reservoirs No. 3 and No. 4. If the maximum output at

Reservoir No. 4 is not to be determined, the quick procedure explained in the foregoing examples is used. Should it be desirable, however, to obtain the maximum output, the maximum is first obtained for the drainage area exclusive of the area above Reservoir No. 3 with the effective storage at Reservoir No. 4 for this area (408,000 acre feet) utilized. The prime flows derived, for Area "A" and the portion of Area "B" below Reservoir No. 3, in order to maximize the output from this area, are then employed to obtain the final maximum output at Reservoir No. 4 by distributing the excess flow capacity over the prime at Reservoir No. 4 to Area "A," to the Area above Reservoir No. 3, and to the remaining portion of Area "B" above Reservoir No. 4. The procedure is the same as in Example No. 1 above, but the operation is more laborious because the excess flow must be apportioned to three areas instead of two as in that example.

General Procedure in Maximizing Output

When Rule No. 1 applies, the excess water capacity over the prime flow at the downstream reservoir is assigned to the various areas above the several reservoirs including the intermediate area between the downstream and upstream reservoirs in such a manner that the output computed with these water capacities is a maximum at the downstream reservoir. Several trials will generally be required. This procedure is the same when the flow characteristics are uniform for all areas or when they vary. If the intermediate area between the upstream and downstream reservoirs is divided into portions having varying flow characteristics, the prime flow for each of these portions to be used in the above operation is first found by applying to the intermediate area, using the storage in the downstream reservoir, the first step for maximizing output explained below for the condition in which Rule No. 2 applies.

When Rule No. 2 applies and the flow characteristics are the same for all areas, the correct output is obtained directly from the output curves of the regulation chart.

When Rule No. 2 applies and the flow characteristics of the several areas vary, two steps are necessary in the maximizing procedure. First, the combined effective storage at the downstream reservoir is allocated to the various areas so that the number of months to maximum drawdown for each area is the same and the output, obtained by using the selected relation between water capacity and prime flow, is a maximum. In the next step the prime flows obtained in the first step are used and the excess water capacity over the prime flow at the downstream reservoir is allocated to the various areas so that the maximum output is obtained at the downstream reservoir.

When a single reservoir regulates flow from two or more areas having varying flow characteristics, the two-step procedure just described is employed to obtain the maximum output.

Need for Obtaining Maximum Output

As stated in Example No. 1, it is doubtful that the work and time involved in finding the maximum output is worthwhile in most instances. If the regulated flows from the several areas do not differ drastically and if the stream flows themselves are not greatly at variance so that the Prime Flow-Water Capacity-Output Curves are similar, the output can be obtained with sufficient accuracy for the purpose without using the maximizing procedures. The results are usually more accurate than the basic data. Without maximizing, the

outputs for the reservoirs in the foregoing examples would have been obtained using water capacities equal to twice the prime flows for each area, as installations having water capacities of twice the prime flows have been assumed for the purposes of this paper. This quick method of obtaining the output has been demonstrated in each example, and application of the method is clearly shown thereby.

If the maximum outputs are not to be obtained, the Output Curves shown on Figures 3 and 4 are more convenient to use than the curves shown on Figure 5. In fact, if a single relationship between water capacity and prime flow is to be used, only one output curve need be derived and drawn. This is true also when maximizing will not be done for conditions where Rule No. 1 applies, and when only the first step of the maximizing process explained in Example No. 4 is to be employed where Rule No. 2 is applicable. When the maximum outputs are to be determined, output curves as shown on Figure No. 5 should be substituted on the regulation charts for the output curves shown on Figures No. 3 and No. 4.

In deriving output curves similar to those on Figure 5, it will often be desirable to use additional water capacities greater than 4 times the prime flow which was the maximum used herein. During the derivation of each curve the water capacity required to utilize all flows of record should be noted. These water capacities when plotted against the total potential output furnish the upper terminal points of the curves. For example, for Area "B" the maximum water capacity required to obtain the total potential output was found to be 6.94 cfs per square mile for prime flows of 0.2, 0.4, 0.6, and 0.8 cfs per square mile and to be 5.56 cfs per square mile for a prime flow of 0.9 cfs per square mile (See these discharges in Table 4).

Economic Installation for Secondary Power

Referring to Figure 5 each curve shows the total kilowatt hours that can be obtained with any water capacity. But the power obtained at a given water capacity is equal to the area of the regulated flow duration curve below that water capacity expressed in kilowatt hours per year per foot of head. Therefore, the output curves on Figure 5 and the regulated flow duration curves are related. The economic installation for secondary power can be readily obtained from the regulated flow duration curve as follows:

$$\frac{dq \times h}{14.6} \times P \times 8760\ V = C\ dkw$$

Where,

dq = differential change in water capacity of the installation.

h = the head

P = Percentage of time the differential change in capacity must operate to be economical (expressed as a decimal).

V = The value per kwh of secondary power.

C = The cost per kilowatt year of additional capacity.

dkw = Differential change in kw capacity corresponding to the change in water capacity.

The figure of 14.6 in the denominator was obtained by assuming an overall efficiency of 80.8%. This efficiency was used throughout this paper for convenience of calculation. No implication that this is the proper efficiency to use is intended.

Reducing to a one foot head, h in the above equation becomes 1.

$$\text{But, } \frac{dq}{14.6} = \text{dkw}$$

$$\text{Therefore, } P \times 8760 \times V = C$$

$$\text{and} \qquad P = \frac{C}{8760V}$$

When this percentage of time is applied to the regulated flow duration curve, the economic water capacity for secondary power is indicated by the ordinate of the duration curve.

Since each curve of Figure 5 represents the area under the regulated flow duration curve, the economic installation for secondary power can be obtained from these curves also. The derivation is as follows:

$\frac{C}{V}$ = the number of hours per year that the last unit of capacity must operate to be economically justified.

$$\frac{dq}{14.6} \times \frac{C}{V} = \text{d kwh}$$

d kwh = kwh/year that can be obtained from the differential capacity corresponding to dq operating for $\frac{C}{V}$ hours per year.

$$\text{Then, } \frac{dq}{dkwh} = \frac{14.6\ V}{C}$$

Since $\frac{dq}{dkwh}$ is the slope of the curves on Figure 5, the point on each curve where the slope is the same as that given by the foregoing formula is the economic installation for secondary power for the reservoir with the storage that produces that output curve. For example:

Assume,

Annual cost per kw of additional installation = \$6 = C

Value per kwh of secondary = \$.003 = V

Then,

$$\frac{dq}{dkwh} = \frac{14.6 \times .003}{6} = \frac{1}{137}$$

The above slope has been shown on Figure 5, and the economic water capacity for secondary power has been indicated on the curves.

Unless the regulated prime flow, or the load factor of the installation is low, the value of secondary power is high, or the regulated flow duration curve is fat and steep for durations below 50% of the time, it is usually not necessary to test for economic installation for secondary. The water capacity obtained by dividing the prime flow by the load factor of the installation will generally exceed the economic water capacity for secondary power.

Flow Duration from the Output Curves

The relation of the output curves shown on Figure 5 to the regulated flow duration curves has been explained above. Since they are related, the flow durations can be obtained from the output curves of Figure 5 by use of the following equations:

$$\frac{dq}{14.6} \times P \times 8760 = dkwh$$

$$\frac{dq}{dkwh} = \frac{1}{600P}$$

Thus, the slope at any point on an output curve of Figure 5 represents the percentage of time that the flow at that point is equalled or exceeded on the regulated flow duration curve. Several slope lines have been drawn on Figure 5 with the duration in percent of time shown thereon. These durations are useful in adjusting the gross output for leakage and evaporation loss.

Adjustment for Leakage and Evaporation Loss

The evaporation and other water losses that occur when the discharge is in excess of the water capacity of the installation do not reduce the output but serve to reduce the spillway discharge only. Thus, if it is determined that the average leakage and evaporation losses equal "q" cubic feet per second, the reduction in gross average annual output in kwh by virtue thereof will be,

$$\frac{q \times h}{14.6} \times 8760 \ (1.00-P)$$

In the case of evaporation loss, it is recognized that the evaporation is not uniform throughout the year, and, accordingly, use of the foregoing formula results in an approximation. This is especially true since periods of spill are usually humid periods also with less evaporation than at other times. But refinement by applying actual rates of evaporation to the various months as they occur is not justified. Use of the above formula to adjust the average annual output for leakage and evaporation loss is satisfactory for all practical purposes. The head, "h," in the formula should be the average head. In computing the average evaporation loss in cubic feet per second, the evaporation rate should be applied to the average reservoir area or to the reservoir area at average drawdown.

The leakage and evaporation not only reduce the total output, but they reduce the prime flow and primary output as well. The estimated leakage can be deducted directly from the gross prime flow. To be as exact as possible in adjusting for evaporation, it would be necessary to compute the evaporation loss during each month of drawdown in the critical low flow period using the evaporation and the average reservoir area for each month. For most investigations, however, it is sufficiently accurate to use the following formula:

$$\text{Evap. loss in prime flow, cfs} = \frac{E}{12 \times 12} \times \frac{A}{60.33}$$

$$= .000115\ EA$$

E = Average annual evaporation in inches.

A = Average reservoir area or reservoir area at average drawdown during critical low flow period, from beginning of drawdown to maximum drawdown.

This loss and other losses are deducted from the gross prime flow, and the net primary kilowatt output is computed from the remainder.

In determining the net outputs for a system of reservoirs, the evaporation losses in cubic feet per second at upstream reservoirs are losses at downstream reservoirs also. Therefore, they must be applied at a downstream reservoir in a manner similar to the evaporation losses from that reservoir.

Head for Output Computations

All power outputs given above were based on a one-foot head. By multiplying by the average annual head and the average head for prime as applicable the gross outputs will be obtained. Study of a number of mass curves and hydrographs utilizing storage that produces a substantial degree of regulation indicates that use of the head at 35% of maximum drawdown is satisfactory for determining total output. Should the degree of regulation be small, the head would be somewhat higher.

With respect to primary output, when the reservoir level is high, since the head is high also, the prime flow as computed herein need not be discharged to produce the primary output. Moreover, the storage is greater in the upper levels of the reservoir than in the lower levels. Accordingly, the reservoir will pass through the upper levels at a slower rate than through the lower levels. Study of drawdowns during various critical periods taking into consideration the slower rates of drawdown at the higher levels has indicated that for general investigations, it will also be satisfactory to use the head at 35% of drawdown for the purpose of determining primary output. This is based on operation to produce a constant primary power output during the critical low flow period. If the drawdown is large, it may not be practicable to select turbines that can conform to this method of operation. Accordingly, the investigator may wish to select a lower head to suit the drawdowns that may require it or to be consonant with the method of plant operation that will be used should it vary from the constant output method.

After the reservoirs in a system, the heights of dam, and the drawdowns have been derived by the foregoing methods, each reservoir in the system can be studied individually to ascertain the applicable heads and the power outputs more closely. However, unless unusual circumstances exist, this is not needed in the general investigation stage of a reservoir system.

CONCLUSION

The regulation chart method of obtaining the prime flow and the power output for a system of reservoirs is a simple procedure when a few basic rules and operations given in this paper are mastered. Many different drawdowns

in a reservoir can be investigated quickly. When a large and comprehensive system of reservoirs and power plants in a river basin must be studied, use of the charts appears to be the only practicable way of arriving at reasonably correct capacities and outputs without an inordinate investment of time and effort.

ACKNOWLEDGMENT

The writer was first introduced to the regulation chart method for hydro electric power computations many years ago by Mr. F. A. Renner, Civil Engineer, Lehighton, Pennsylvania. At that time, the regulation chart was in rather primitive form, and the details of its proper use were not understood. Together, Mr. Renner and the writer improved the chart, and through considerable study discovered proper methods of its preparation and use. A short time thereafter Mr. Renner wrote a substantial volume, entitled "Planning the Power Development of Rivers," which included preparation and use of the regulation chart as well as many other items involved in hydro electric investigations. Unfortunately, this volume was never published. It was mimeographed, however, so that a number of interested persons, especially in the Corps of Engineers, have copies.

Since Mr. Renner wrote his volume, the writer has made substantial changes in the regulation chart and in its application. These have improved the accuracy of the results, so that regulated flows and power outputs can be obtained from the regulation chart that, within the limits of accuracy of the reading of the curves, are as correct as can be obtained through manual regulation of the stream flows. Many of these changes are presented for the first time in this paper.

As implied by the foregoing, the writer is deeply indebted to Mr. F. A. Renner.

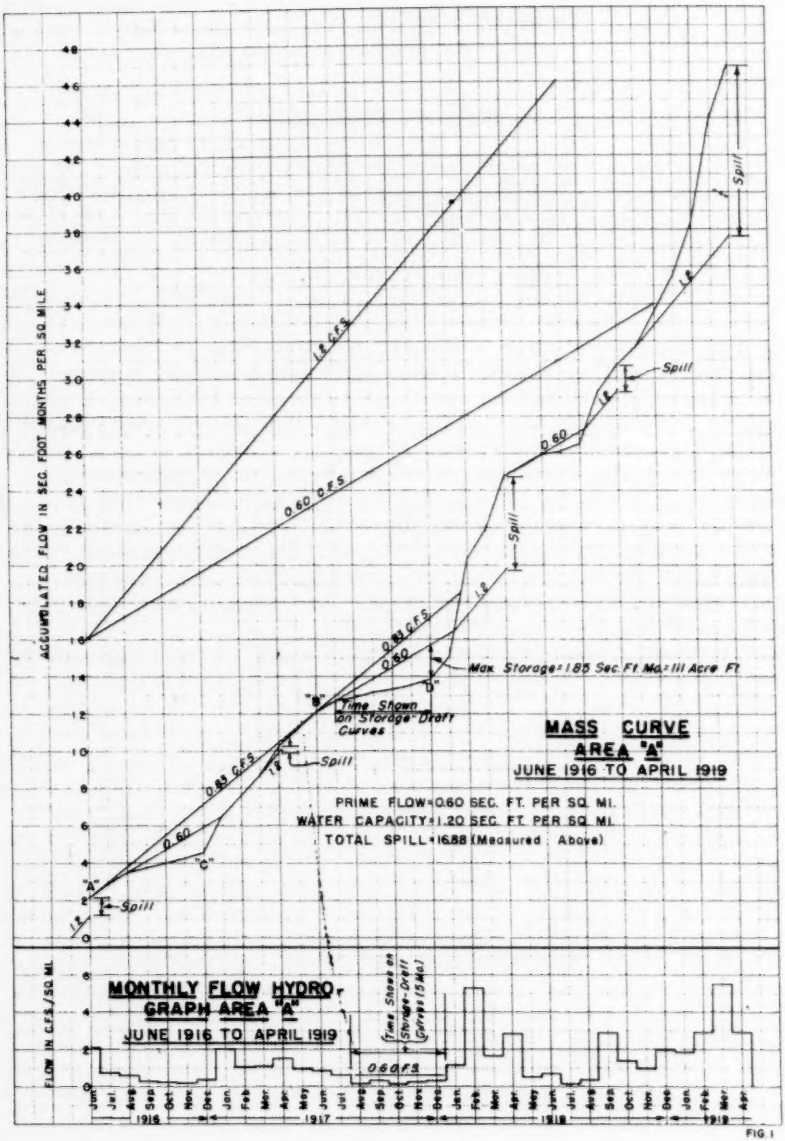
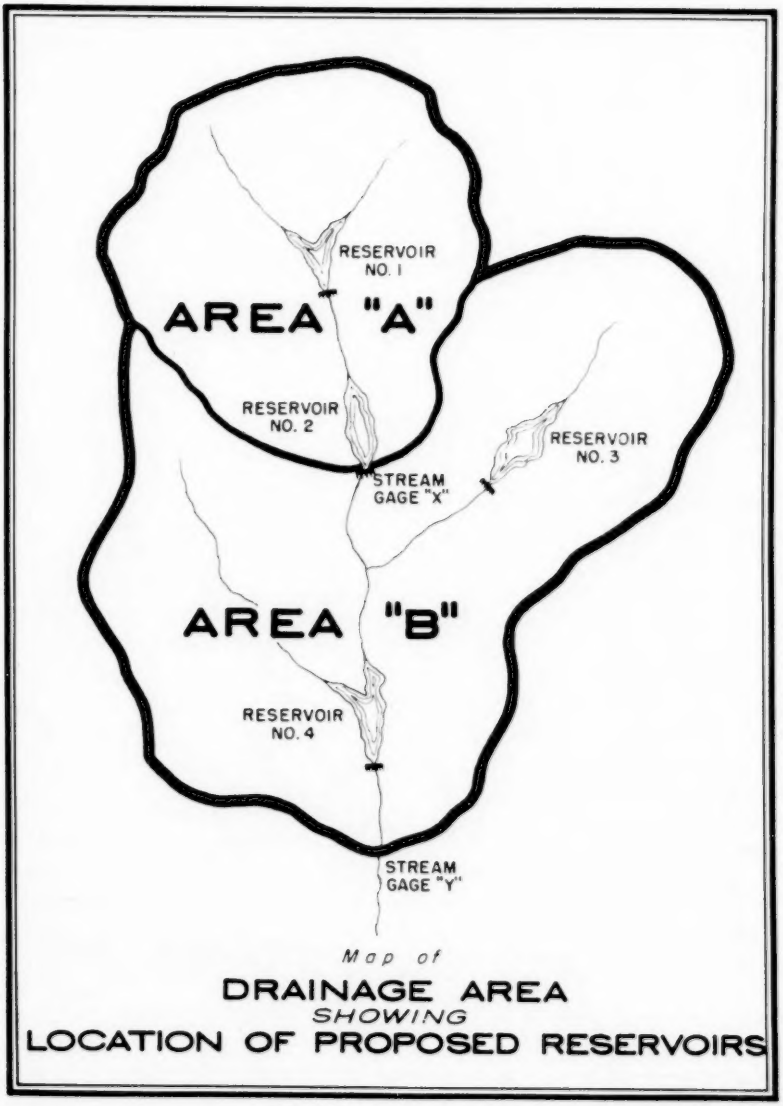

ACCUMULATED FLOW IN SEC. FOOT MONTHS PER SQ. MILE
1.2 C.F.S.
0.60 C.F.S.
0.83 C.F.S.
Spill
Max. Storage = 185 Sec. Ft. Mo. = 111 Acre Ft.
Time Shown on Storage Draft Curves
"A"
"B"
"C"
"D"
MASS CURVE
AREA "A"
JUNE 1916 TO APRIL 1919
PRIME FLOW = 0.60 SEC. FT. PER SQ. MI.
WATER CAPACITY = 1.20 SEC. FT. PER SQ. MI.
TOTAL SPILL = 16.88 (Measured Above)
FLOW IN C.F.S./SQ. MI.
MONTHLY FLOW HYDRO-GRAPH AREA "A"
JUNE 1916 TO APRIL 1919
(Time Shown on Storage-Draft Curves 15 Mo.)
0.60 C.F.S.
1916
1917
1918
1919

FIG 1



Map of
DRAINAGE AREA
SHOWING
LOCATION OF PROPOSED RESERVOIRS

FIG. 2

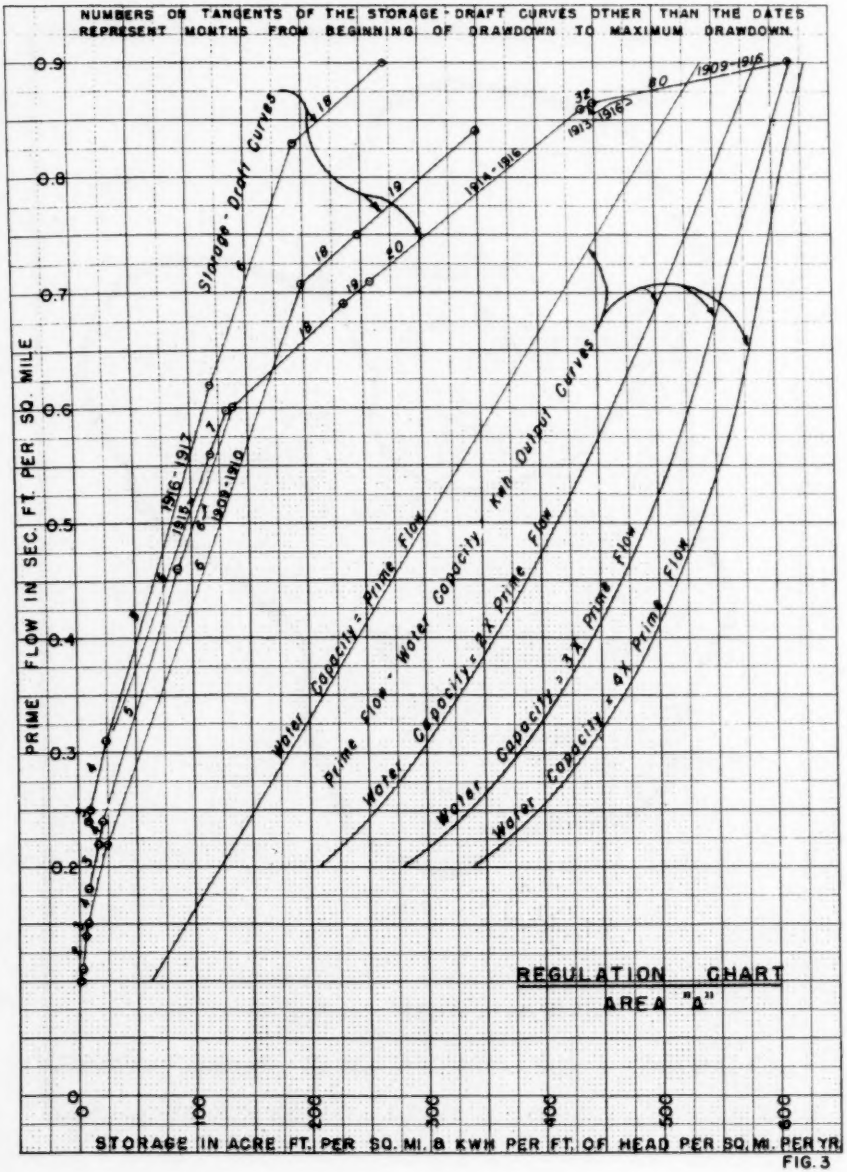

NUMBERS ON TANGENTS OF THE STORAGE-DRAFT CURVES OTHER THAN THE DATES REPRESENT MONTHS FROM BEGINNING OF DRAWDOWN TO MAXIMUM DRAWDOWN.
PRIME FLOW IN SEC. FT. PER SQ. MILE
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0
Sloodge - Draft Curves
1916-1917
1915-1916
1909-1910
1914-1916
1909-1916
Water Capacity = Prime Flow
Prime Flow - Water Capacity + Kwh Output Curves
Water Capacity = 2X Prime Flow
Water Capacity = 3X Prime Flow
Water Capacity = 4X Prime Flow
REGULATION CHART
AREA "A"
0
100
200
300
400
500
600
STORAGE IN ACRE FT. PER SQ. MI. & KWH PER FT. OF HEAD PER SQ. MI. PER YR.

FIG. 3

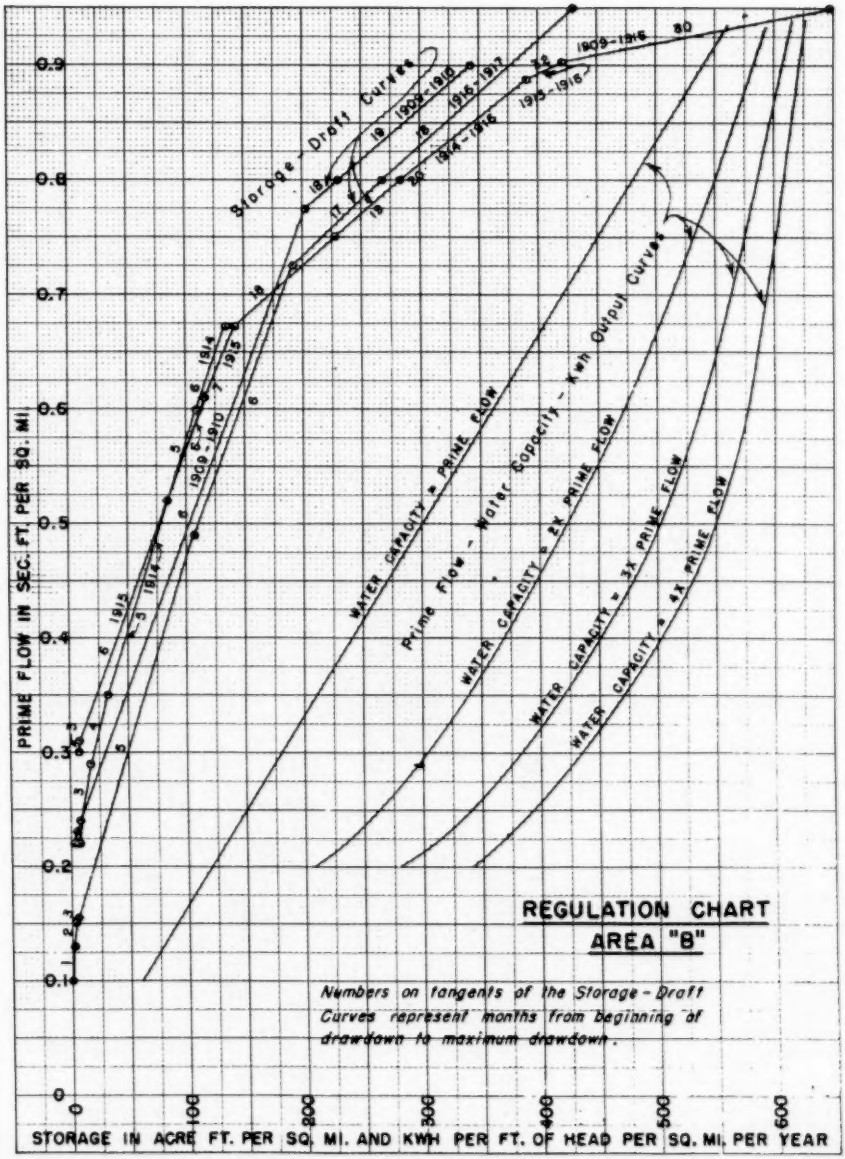

PRIME FLOW IN SEC. FT. PER SQ. MI.
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
100
200
300
400
500
600
STORAGE IN ACRE FT. PER SQ. MI. AND KWH PER FT. OF HEAD PER SQ. MI. PER YEAR
Storage - Draft Curves
1909-1916
1908-1910
1915-1917
1914-1916
1913-1916
WATER CAPACITY = PRIME FLOW
Prime Flow - Water Capacity - kwh Output Curves
WATER CAPACITY = 2X PRIME FLOW
WATER CAPACITY = 3X PRIME FLOW
WATER CAPACITY = 4X PRIME FLOW
REGULATION CHART
AREA "8"
Numbers on tangents of the Storage - Draft Curves represent months from beginning of drawdown to maximum drawdown.

FIG. 4

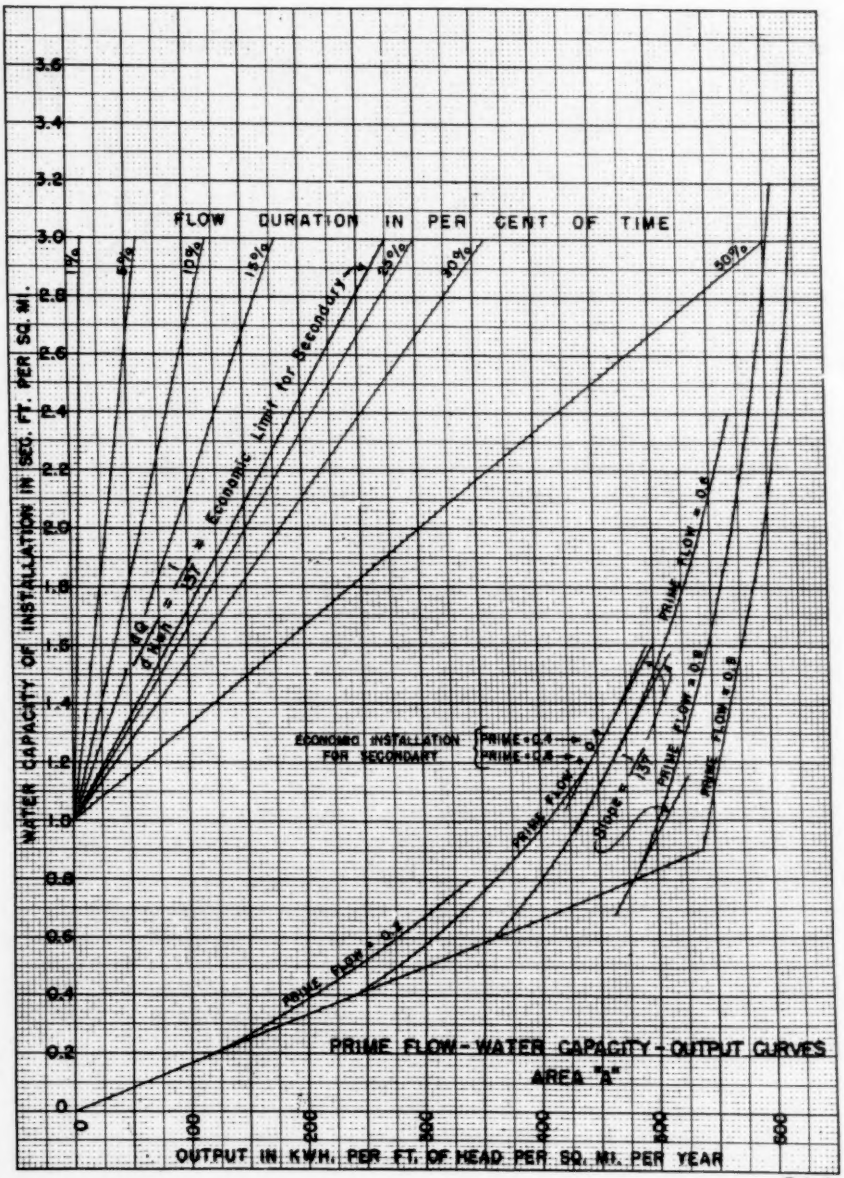

FLOW DURATION IN PER CENT OF TIME
1%
5%
10%
15%
20%
30%
50%
WATER CAPACITY OF INSTALLATION IN SEC. FT. PER SQ. MI.
3.6
3.4
3.2
3.0
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
Economic Limit for Secondary
ECONOMIC INSTALLATION FOR SECONDARY
PRIME=0.4
PRIME=0.6
PRIME FLOW = 0.4
PRIME FLOW = 0.6
PRIME FLOW = 0.8
PRIME FLOW = 0.8
PRIME FLOW = 0.4
Slope = 1/5
PRIME FLOW - WATER CAPACITY - OUTPUT CURVES
AREA "A"
0
100
200
300
400
500
600
OUTPUT IN KWH. PER FT. OF HEAD PER SQ. MI. PER YEAR

FIG. 5

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

VOLUME 82 (1956)

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092 (HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103 (IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c], 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

VOLUME 83 (1957)

JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158 (EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246 (HW2), 1247(HW2), 1248(WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257(IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267 (PO3), 1268(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275 (SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283 (HY3)[c],1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297(HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303 (ST4), 1304(ST4) 1305(SU1), 1306(SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312 (ST4), 1313(ST4), 1314(ST4), 1315(ST4), 1316(ST4), 1317(ST4), 1318 (ST4), 1319(SM3), 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324(AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338(SA4), 1339(CO1) 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347(BD1), 1348(HY4)[c], 1349(SA4)[c], 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360(IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369(WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378(HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395(HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404(HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413(EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417 (HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

c. Discussion of several papers, grouped by Divisions.

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